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FINAL

BASEWIDE REMEDIATION ASSESSMENT GROUNDWATER STUDY (BRAGS) MARINE CORPS BASE CAMP LEJEUNE, NORTH CAROLINA

CONTRACT TASK ORDER 0140

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LIST OF ACRONYMS AND ABBREVIATIONS

AFCEE	Air Force Center for Environmental Excellence
ARM	Absolute Residual Mean (see MAE)
b	Aquitard Thickness
BC	Boundary Conditions (MODFLOW)
BRAGS	Basewide Remediation Assessment Groundwater Study
C_{drn}	Drain Cell Conductance (MODFLOW)
C_{riv}	River Cell Conductance (MODFLOW)
CA	Corrective Action
CCLs	Ceiling Concentration Limits
CCP	Central Coastal Plain
CERCLA	Comprehensive Environmental Response, Compensation, and Liability Act (Superfund)
cfd	Cubic Feet per Day
cis-1,2-DCE	cis-1,2-dichloroethene
cm/sec	Centimeters per Second
CMS	Corrective Measure Study
CTO	Contract Task Order
DCE	cis and trans-1,2-dichloroethene
DNAPLs	Dense Non-Aqueous Phase Liquids
DoN	Department of the Navy
ESE	Environmental Science and Engineering, Inc.
FFA	Federal Facilities Agreement
ft^2/day	Square Feet/Day (unit of transmissivity)
ft/day-s	Feet/Day (unit of velocity or hydraulic conductivity)
ft/day/ft	Feet/Day/Feet (unit of leakance)
FS	Feasibility Study
G3CTM	G-3 Contaminant Transport Model (for groundwater discharge to surface water -- NC DENR)
gpm	Gallons per Minute
GWQ	Groundwater Quality
HFB	Horizontal Flow Barrier (MODFLOW)
HPIA	Hadnot Point Industrial Area
IP	Implementation Plan
IR	Installation Restoration
K	Hydraulic Conductivity (ft/day)
K_{ii}	Hydraulic Conductivity Tensor in the i direction (MODFLOW)
K_v	Vertical Hydraulic Conductivity (ft/day)

L	Length of Cell (MODFLOW)
m	Thickness of River or Stream Sediments (MODFLOW)
ME	Mean Error
MAE	Mean Absolute Error
MCB	Marine Corps Base
mi^2	Square Mile
MSL	Mean Sea Level
n	Sample Size
NC	North Carolina
NC DENR	North Carolina Department of Environment and Natural Resources
NPL	National Priorities List
OU	Operable Unit
PCE	Tetrachloroethene (perchloroethylene)
ppb	Parts per Billion
RASA	Regional Aquifer System Analysis
RCRA	Resource Conservation and Recovery Act
RF	Retardation Factors
RI	Remedial Investigation
RI/FS	Remedial Investigation/Feasibility Study
RM	Residual Mean (see ME)
RMS	Root Mean Square (see RMSE)
RMSE	Root Mean Square Error
RSD	Residual Standard Deviation (see SDE)
SDE	Standard Deviation of the Errors
SMP	Site Management Plan
TCE	Trichloroethene
trans-1,2-DCE	Trans-1,2-dichloroethene
USEPA	United States Environmental Protection Agency
USGS	United States Geological Survey
UST	Underground Storage Tank
VC	Vinyl Chloride
VOC	Volatile Organic Compound
W	Width of Cell (MODFLOW)
WAR	Water and Air Research, Inc.
X	X Axis of Cartesian Coordinates associated with Hydraulic Conductivity (MODFLOW)

Y	Y Axis of Cartesian Coordinates associated with Hydraulic Conductivity (MODFLOW)
Z	Z Axis of Cartesian Coordinates associated with Hydraulic Conductivity (MODFLOW)
°F	Degrees Fahrenheit

EXECUTIVE SUMMARY

Marine Corps Base (MCB), Camp Lejeune, bisected by the tidal estuary New River, borders the Atlantic Ocean and encompasses approximately 236 square miles of the Atlantic Coastal Plain of North Carolina. Ongoing management of water supply withdrawals and evaluations of potential impacts to groundwater due to remediation are required. Although regional geologic and hydrogeologic studies have been conducted by the United States Geological Survey (USGS) at MCB, Camp Lejeune (Harned, et al, 1989 and Cardinell, et al, 1993), the resulting effects of these remedial groundwater pump and treatment systems on the underlying aquifers have not been evaluated. Existing USGS Regional Aquifer System Analysis (RASA) and North Carolina Department of Environment, and Natural Resources (NC DENR) groundwater flow models were evaluated; however, over-large scales and lack of detail in the impacted surficial units precluded their use at MCB, Camp Lejeune.

The Fiscal Year 1998 Site Management Plan (SMP) for MCB, Camp Lejeune, the primary document referenced in the FFA, identifies 42 sites that require Remedial Investigation/Feasibility Study (RI/FS) activities. In addition to the RI/FS sites, 135 underground storage tank (UST) sites have also been identified, 126 of which have undergone environmental investigations. Based on information obtained from the SMP and from Base personnel, 28 groundwater remediation systems (i.e., groundwater pumping and treatment or air sparging) are currently operating, waiting construction, or under consideration.

Consequently, the focus of this study is to develop a Basewide groundwater flow model which can be used to evaluate the effects of various groundwater remediation projects under the auspices of the Basewide Remediation Assessment Groundwater Study (BRAGS) that are active or planned for MCB, Camp Lejeune. Two three-dimensional groundwater flow models were developed for use at MCB, Camp Lejeune: a comprehensive Basewide model (referred to herein as the BRAGS model) and a site-specific model for Site 82 (Piney Green VOC Area). The BRAGS model was constructed first based on composite groundwater elevation data taken from 30 IRP/UST sites at the Base and from published USGS data collected from the water supply wells at the Base. The site-specific model was then constructed on the foundation laid by the BRAGS model using data primarily from IRP Sites 82, 6, 9, and 3, UST Site 889-891, and from the nearby water supply wells. Both models were constructed with MODFLOW (a finite-difference numerical flow model) and calibrated to measured head data collected by Baker from 1992 to 1993. MODPATH was used to generate particle pathlines based on the results of MODFLOW.

The objectives of this modeling effort were to provide LANTDIV and MCB, Camp Lejeune with one or more working groundwater flow models that can be used to:

- Describe how groundwater flows beneath the entire Base as well as under individual sites of concern.
- Demonstrate the effects of groundwater withdrawals (supply and remedial) on the aquifers in question (most notably the surficial unit and the Castle Hayne Aquifer).
- Predict the relative effectiveness of various remediation schemes at individual sites (including Site 82).

As "working" models, it is imperative that these groundwater flow models be updated as new information becomes available. Only updated models will be effective decision-making tools to optimize groundwater resource management, protection, and restoration. It is envisioned that personnel at LANTDIV or Camp Lejeune (or their representatives) will update and use these models to determine the relative effectiveness of various remedial scenarios at individual sites around the Base.

The BRAGS model was constructed first so that the conceptual model of the entire Base could be tested. After the BRAGS model was calibrated, the model for Site 82 was constructed. This enabled the use of the previously calibrated inputs to be used, with some adjustments, at the site level of detail. As new information became available during the course of the study, the BRAGS model was updated and recalibrated. The update incorporated the new data from the Site 82 model (including the results of the pumping test) and from the Site 73 model into the BRAGS model.

The BRAGS groundwater flow model presented herein portrays the three-dimensional pattern of groundwater flow within the surficial unit and the Castle Hayne Aquifer. The BRAGS model predicts the elevation and flow direction of the surficial and Castle Hayne groundwater in many areas around the Base where no data currently exist. The BRAGS model also demonstrates that discharge to the New River and its tributaries is the controlling factor on flow directions in the Castle Hayne Aquifer in the vicinity of Camp Lejeune. The model output indicated that the relatively high-volume withdrawal rates of the supply wells have a localized effect on the water levels in the Castle Hayne; however, large numbers of actively pumping wells in small areas have the potential to induce saltwater intrusion into the upper Castle Hayne Aquifer. This effect is most pronounced in the Paradise Point area along Brewster Boulevard. To minimize drawdown and the resulting potential for saltwater intrusion, actively pumping water supply wells should not be grouped together in small areas but should be spread out in a line perpendicular to the ambient flow direction (not parallel to it).

One of the concerns that initiated this modeling effort was that the potential number of pump and treat remedial actions at the Base may negatively impact the supply of available groundwater. The BRAGS model strongly indicated that the low volumes of water withdrawn from the surficial unit and/or the Castle Hayne Aquifer during such remedial actions will not measurably affect the groundwater supply at the Base. The results of the Site 82 model corroborate this theory.

The Site 82 model describes the three-dimensional pattern of groundwater flow in the surficial unit and Castle Hayne Aquifer. The Site 82 model demonstrates the effects of proposed remedial groundwater withdrawals on the surficial unit and the Castle Hayne Aquifer. The model also demonstrates that the relatively low-volume withdrawal rates of the extraction wells will have a localized effect on the water levels in the surficial unit and the Castle Hayne Aquifer.

The Site 82 model directly addressed the third objective: it clearly showed the relative effectiveness of various site-specific remediation schemes. The locations of the extraction wells in the surficial and in the Castle Hayne Aquifer were finalized by the successful running of the model. "Success" was indicated by complete hydraulic control or "capture" of the contaminant plume. Also, the model indicated that the low volumes of water withdrawn during remedial actions in the surficial unit or the upper Castle Hayne will not measurably affect the groundwater supply at the Base.

The groundwater flow models described herein will be useful in managing the future RI activities at the Base. The BRAGS model will be especially useful for determining the groundwater flow patterns in areas where no data currently exists and it gives a regional perspective on site-specific modeling. Future groundwater flow and/or contaminant transport modeling done at the site level should be coordinated with the BRAGS groundwater flow model so that the "big picture" of the groundwater flow is consistent across the Base.

BASEWIDE REMEDIATION ASSESSMENT GROUNDWATER STUDY (BRAGS)
Marine Corps Base, Camp Lejeune, North Carolina

1.0 INTRODUCTION

Marine Corps Base (MCB), Camp Lejeune was placed on the Comprehensive Environmental Response, Compensation, and Liability Act (CERCLA) National Priorities List (NPL) on October 4, 1989 (54 Federal Register 41015, October 4, 1989). Subsequent to this listing, the United States Environmental Protection Agency (USEPA) Region IV, the North Carolina Department of Environment and Natural Resources (NC DENR), and the United States Department of the Navy (DoN) entered into a Federal Facilities Agreement (FFA) to conduct remedial investigations at MCB, Camp Lejeune. The primary purpose of the FFA is to ensure that environmental impacts associated with past and present activities at MCB, Camp Lejeune are thoroughly investigated and appropriate CERCLA response/Resource Conservation and Recovery Act (RCRA) corrective action alternatives are developed and implemented, as necessary, to protect public health, welfare, and the environment (FFA, 1989).

The Fiscal Year 1998 Site Management Plan (SMP) for MCB, Camp Lejeune, the primary document referenced in the FFA, identifies 42 sites that require Remedial Investigation/Feasibility Study (RI/FS) activities. In addition to these sites, 135 underground storage tank (UST) sites have also been identified, of which 126 sites have undergone environmental investigations. Based on information obtained from the SMP and from Base personnel, more than 28 of these IRP/UST sites are currently undergoing or are proposed for groundwater remediation actions (i.e., groundwater pumping and treatment or air sparging). Although regional geologic and hydrogeologic studies have been conducted by the United States Geological Survey (USGS) at MCB, Camp Lejeune (Harned, et al, 1989 and Cardinell, et al, 1993), the resulting effects of these groundwater pump and treatment systems on the underlying aquifers have not been evaluated. Consequently, the focus of this study is to develop a Basewide groundwater flow model which can be used to evaluate the effects of various groundwater remediation projects under the auspices of the Basewide Remediation Assessment Groundwater Study (BRAGS) that are active or planned for MCB, Camp Lejeune.

In the course of this modeling effort, two working groundwater flow models were developed for use at MCB, Camp Lejeune: a comprehensive Basewide model and a site-specific model for Site 82 (Piney Green VOC Area). The BRAGS model was constructed first based on groundwater elevation data taken from 30 sites at the Base and from USGS data collected from the water supply wells at the Base. The site-specific model was constructed on the foundation laid by the BRAGS model using data primarily from IRP Sites 82, 6, 9, and 3, UST Site 889-891, and from the nearby water supply wells. Both models were calibrated to measured head data collected by Baker from 1992 to 1993.

1.1 Modeling Objectives

The objectives of this modeling effort were to provide the Atlantic Division, Naval Facilities Engineering Command (LANTDIV) and MCB, Camp Lejeune with one or more working groundwater flow models that can be used to:

1. Describe how groundwater flows beneath the entire Base as well as under individual sites of concern;

2. Demonstrate the effects of groundwater withdrawals (supply and remedial) on the aquifers in question (most notably the surficial unit and the Castle Hayne Aquifer); and,

3. Predict the relative effectiveness of various remediation schemes at individual sites (including Site 82).

As "working" models, it is imperative that these groundwater flow models be updated as new information becomes available. Only updated models will be effective decision-making tools for optimal groundwater resource management, protection, and restoration. It is envisioned that personnel at LANTDIV or Camp Lejeune will update and use these models to determine the relative effectiveness of various remedial scenarios at individual sites around the Base.

The BRAGS model was constructed first so that the conceptual model of the entire Base could be tested. After the BRAGS model was calibrated, the model for Site 82 was constructed. This enabled the use of the previously calibrated inputs to be used, with some adjustments, at the site level of detail. As new information became available during the course of the study, the BRAGS model was updated and recalibrated as necessary. The update incorporated the new data from the Site 82 pumping test into the BRAGS model.

1.2 Report Organization

The Final BRAGS report is comprised of one text volume which includes appendices of data from previously conducted pump tests at the Base as well as electronic input and output from the groundwater flow models (provided in Appendices B and C on CD-ROM). The section headings included within this text volume are as follows:

- Previous Investigations and Computer Simulations - Section 2.0
- Hydrogeology of the Camp Lejeune Area - Section 3.0
- BRAGS Groundwater Flow Model - Section 4.0
- Site 82 Groundwater Flow Model - Section 5.0
- Conclusions and Recommendations - Section 6.0
- References - Section 7.0
- Pumping Test Data from Site 82 - Appendix A
- BRAGS Model Data - Appendix B
- Site 82 Model Data - Appendix C

1.3 Location and Setting

MCB, Camp Lejeune is located on the Atlantic Coastal Plain of North Carolina in Onslow County. The Base is in the Tidewater (i.e., tidally-influenced) Region of the North Carolina Coastal Plain (Stuckey, 1965). The facility encompasses approximately 236 square miles and is bisected by the New River. The New River flows in a southerly direction through Camp Lejeune and forms a large, meandering estuary before entering the Atlantic Ocean. The southeastern border of Camp Lejeune is the Atlantic Ocean shoreline. The western and northeastern boundaries of the facility are U.S. Route 17 and State Route 24, respectively. The City of Jacksonville borders Camp Lejeune to the north (see Figure 1-1). Figure 1-1 also shows the locations of the IRP Sites within each Operable Unit (OU) and Figure 1-2 shows the locations of the UST Sites around the Base.

1.4 History

Construction of MCB, Camp Lejeune began in April 1941 at the Hadnot Point Industrial Area (HPIA), where major functions of the Base are located today. The MCB, Camp Lejeune complex, designed to be the "World's Most Complete Amphibious Training Base," consists of 12 general geographical locations under the jurisdiction of the Base Command. These areas include Hadnot Point, Paradise Point, Berkeley Manor/Watkins, Midway Park, Tarawa Terrace I and II, Knox Trailer, French Creek, Courthouse Bay, Onslow Beach, Rifle Range, Camp Geiger, and Montford Point. Table 1-1 lists the acreage in each geographical area of different types of land utilization (e.g., training, operations, storage, administration, etc.).

1.5 Topography

Elevations on the Base vary from sea level to 72 feet above the National Geodetic Vertical Datum of 1929 (hereafter referred to as "mean sea level" or msl); however, most of MCB, Camp Lejeune is between 20 and 40 feet above msl. Drainage at MCB, Camp Lejeune is generally toward the New River, except in areas near the coast where flow is into the Intracoastal Waterway that lies between the mainland and barrier islands. In developed areas of the facility, natural drainage has been altered by asphalt cover (i.e., roadway and parking areas), storm sewers, and drainage ditches. Approximately 70 percent of MCB, Camp Lejeune is comprised of broad, flat interstream areas with poor drainage (WAR, 1983).

1.6 Surface Water Hydrology

The dominant surface water feature at MCB, Camp Lejeune is the New River. It receives drainage from a majority of the Base. The New River is short, with a course of approximately 50 miles on the central Coastal Plain of North Carolina. Upstream from Camp Lejeune and over most of its length, the New River is confined to a relatively narrow channel in Eocene and Oligocene limestones. South of Jacksonville, the river widens dramatically as it flows across less resistant sands, clays, and marls. At MCB, Camp Lejeune, the New River flows in a southerly direction into the Atlantic Ocean through the New River Inlet. Several small coastal creeks drain the area of Camp Lejeune not associated with the New River and its tributaries. These creeks flow into the Intracoastal Waterway, which is connected to the Atlantic Ocean by Bear Inlet, Brown's Inlet, and the New River Inlet. The New River, the Intracoastal Waterway, and the Atlantic Ocean converge at the New River Inlet.

SECTION 1.0 TABLES

TABLE 1-1

LAND UTILIZATION WITHIN DEVELOPED AREAS OF MCB, CAMP LEJEUNE
BRAGS, CTO-0140
MCB, CAMP LEJEUNE, NORTH CAROLINA

Geographic Area	Operation	Training (Instruc.)	Maintenance	Supply/ Storage	Medical	Admin-istration	Family Housing	Troop Housing	CM	CO	Recreation	Utility	Total
Hadnot Point	31 (2.9)	15 (1.4)	154 (14.3)	157 (14.4)	10 (0.9)	122 (11.3)	22 (2.0)	196 (18.1)	115 (10.7)	36 (3.3)	182 (16.9)	40 (3.7)	1,080 (100)
Paradise Point	1 (0)		3 (0.4)	1 (0)			343 (34)	19 (1.9)	31 (3.1)		610 (60.4)	2 (0.2)	1,010 (100)
Berkeley Manor/ Watkins							406 (80)		41 (8.1)	1 (0.2)	57 (11.2)	2 (0.5)	507 (100)
Midway Park		1 (0.4)		2 (0.7)		2 (0.7)	248 (92.2)		8 (3.0)	3 (1.1)	4 (1.5)	1 (0.4)	269 (100)
Tarawa Terrace I and II			3 (0.5)			1 (0.3)	428 (77.4)		55 (9.9)	11 (2.0)	47 (8.5)	8 (1.4)	553 (100)
Knox Trailer							57 (100)						
French Creek	8 (1.4)	1 (0.2)	74 (12.7)	266 (45.6)	3 (0.5)	7 (1.2)		122 (20.9)	22 (3.8)	6 (1.0)	74 (12.7)		583 (100)
Courthouse Bay		73 (28.6)	28 (10.9)	14 (5.5)		12 (4.7)	12 (4.7)	43 (16.9)	15 (5.9)	4 (1.6)	43 (16.9)	11 (4.3)	255 (100)
Onslow Beach	6 (9.8)	1 (1.6)	3 (4.8)	2 (3.2)	1 (1.6)	2 (3.2)		2 (3.2)	12 (19.3)		25 (40.3)	8 (13.0)	62 (100)
Rifle Range		1 (1.3)	1 (1.3)	7 (8.8)	1 (1.3)	5 (6.3)	7 (8.8)	30 (37.5)	5 (6.3)	1 (1.3)	9 (11.3)	13 (16.3)	80 (100)
Camp Geiger	4 (1.9)	15 (6.9)	19 (8.8)	50 (23.1)		23 (10.6)		54 (25.0)	27 (12.5)	2 (1.0)	16 (7.4)	6 (2.8)	216 (100)
Montford Point	6 (2.6)	48 (20.5)	2 (0.9)	4 (1.7)	2 (0.9)	9 (3.9)		82 (35.2)	20 (8.6)	1 (0.4)	49 (21.0)	10 (4.3)	233 (100)
Base-Wide Misc.	1 (0.8)			87 (68.0)		3 (2.3)			19 (14.8)			18 (14.1)	128 (100)
TOTAL	**57 (1.)**	**155 (3.1)**	**287 (5.7)**	**590 (11.7)**	**17 (0.38)**	**186 (3.7)**	**1,523 (30.2)**	**548 (10.8)**	**370 (7.4)**	**65 (1.3)**	**1,116 (22.2)**	**119 (2.4)**	**5,033 (100)**

Notes:

Numbers without parentheses represent total acres.
Numbers within parentheses represent percentage of total acres.
Source: Master Plan, 1988

SECTION 1.0 FIGURES

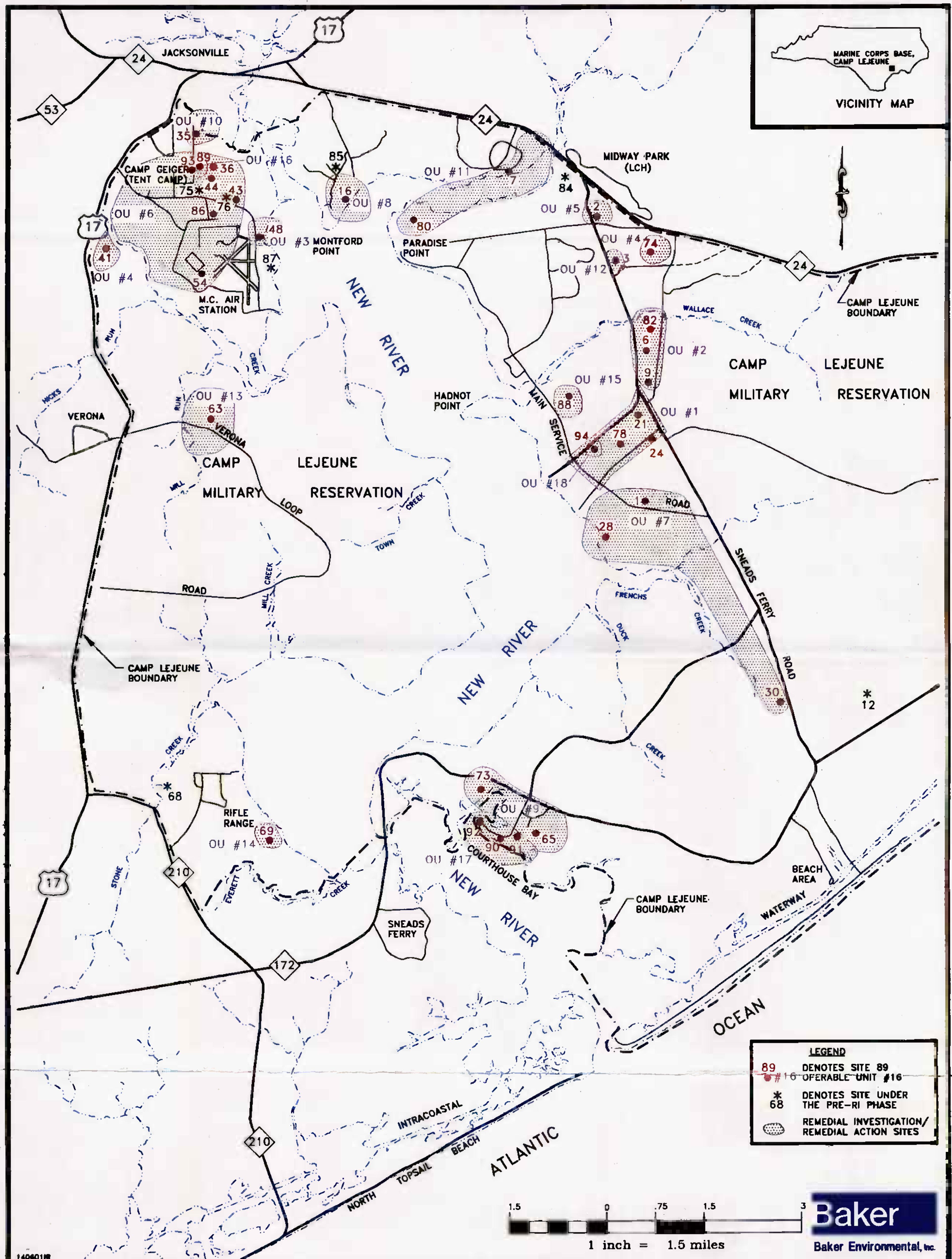


FIGURE 1-1
IRP OPERABLE UNITS AND SITE LOCATION
BRAGS, CTO-0140

MARINE CORPS BASE, CAMP LEJEUNE
NORTH CAROLINA

02013AAB1Y

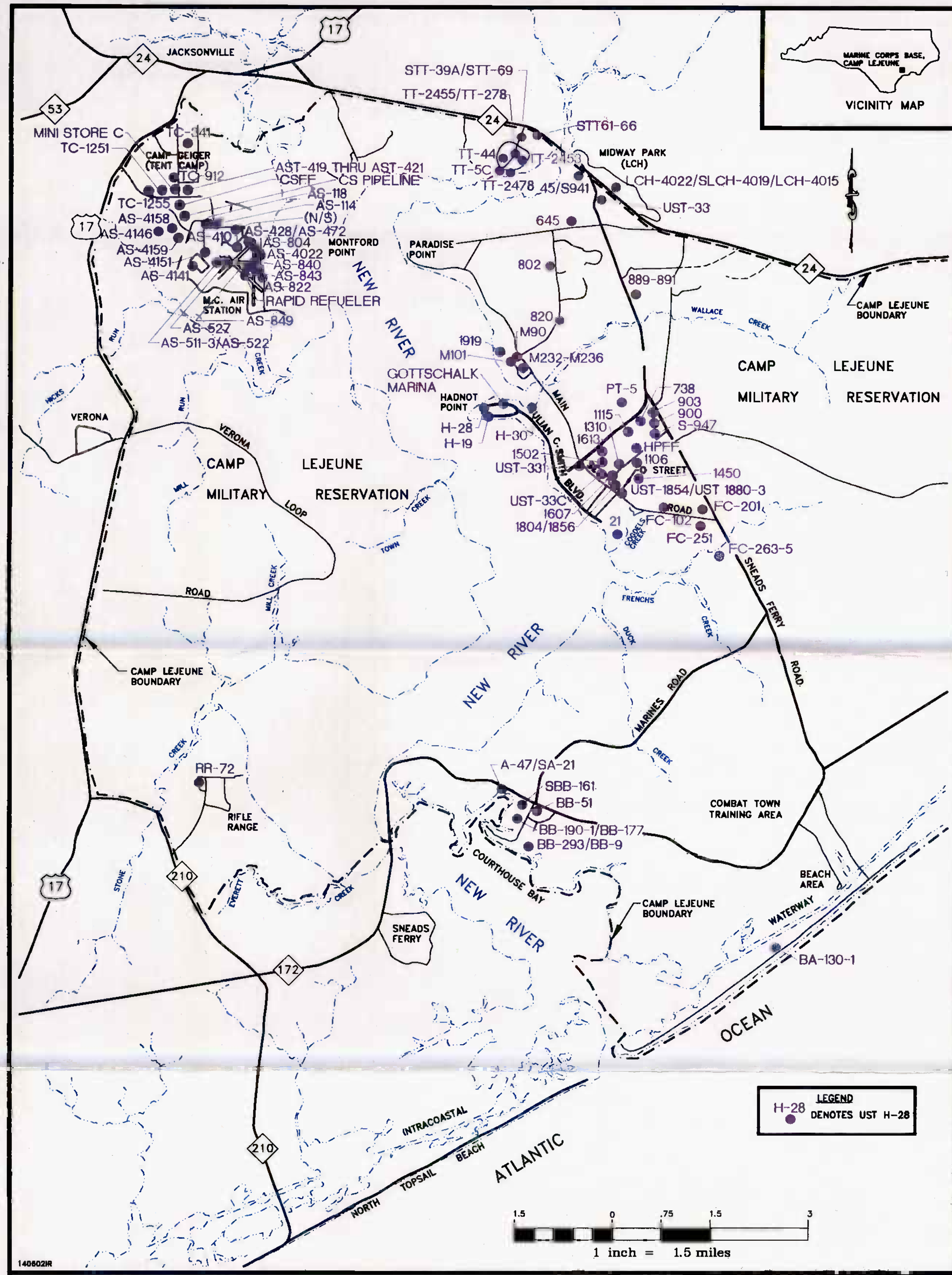


FIGURE 1-2
UST SITE LOCATIONS
BRAGS, CTO-0140

MARINE CORPS BASE, CAMP LEJEUNE
NORTH CAROLINA

2.0 PREVIOUS INVESTIGATIONS AND COMPUTER SIMULATIONS

Over 160 IRP/UST investigations have been conducted regarding the hydrogeological characteristics of the subsurface at and near MCB, Camp Lejeune. The geology and hydrogeology of the region and of the area adjacent to Camp Lejeune has been described by the USGS in several recent reports from 1989 to 1993. Of particular pertinence to this effort were the publications by Cardinell et al (1993), Geise et al (1991), Winner and Coble (1989), and Harned et al (1989). On-site investigative activities conducted by Baker and other firms have added to the existing data with regard to the near-surface geology and hydrogeology.

At least three groundwater flow models have been constructed of the region encompassing the Base. At the outset of this effort it was thought that one or more of the existing regional groundwater flow models may be adapted for use on a smaller scale. The Regional Aquifer System Analysis (RASA) program generated two regional groundwater flow models and the North Carolina Geological Survey created a model of the Central Coastal Plain (CCP). These three existing models were examined and it was subsequently determined that the they were too large in scale and not detailed enough to yield meaningful results for use at MCB, Camp Lejeune. A brief description of each (and the reasons for its unsuitability to the task at hand) follows:

- One of the RASA models (Leahy & Martin, 1993) encompassed the entire area of the Northern Atlantic Coastal Plain (from Long Island, New York to North Carolina). The scale of this model was much too large to be adapted for use at such a comparatively small area like MCB, Camp Lejeune (which fit into one of the cells of the RASA model's finite difference grid).

- Another RASA model (Geise, G.L., J.L. Eimers, & R.W. Coble, 1991) covered only the Atlantic Coastal Plain in North Carolina. Unfortunately, the scale of this model was also too large to be used directly because the area of MCB, Camp Lejeune took up only 4 cells wide by 4 cells long in the model grid. However, the inputs to this model were used extensively as background information for the BRAGS model at MCB, Camp Lejeune.

- The CCP model (Eimers, J.L., W.L. Lyke, & A.R. Brockman, 1990) covered a smaller region within North Carolina (MCB, Camp Lejeune was encompassed by approximately 10 cells wide by 10 cells long). However, the CCP modeled only the Cretaceous Peedee Aquifer and below. The surficial unit and the Castle Hayne Aquifer were not specifically modeled. It is possible that the current modeling effort could be used to generate input parameters in a version of the CCP model for the surficial unit and the Castle Hayne Aquifer. That, however, is beyond the scope of this modeling effort, but may be of interest to water management officials in the future.

None of these existing groundwater flow models dealt with either the surficial unit or the Castle Hayne Aquifer in a meaningful manner over the area of interest. Because these two potentially vulnerable hydrologic units were of paramount importance in this study, a new model was deemed necessary.

3.0 HYDROGEOLOGY OF THE CAMP LEJEUNE AREA

3.1 Physiography

MCB, Camp Lejeune lies within the Tidewater region of the Atlantic Coastal Plain physiographic province (Stuckey, 1965). The Atlantic Coastal Plain is an eastward-thickening wedge of sediments lying atop the basement of Precambrian bedrock. This wedge varies from a thickness of zero near the Fall Line to more than 10,000 feet near and under the Atlantic Ocean (Winner & Coble, 1989). The Tidewater region is the portion of the Atlantic Coastal Plain that is influenced by diurnal ocean tides and is generally low-lying, swampy terrain with elevations ranging from sea level to about 50 ft.

3.2 Geologic and Hydrogeologic Framework

Beneath Camp Lejeune are seven water-bearing units, each comprised of one or more formations: an unnamed surficial unit of recent and Pleistocene age, the Castle Hayne Aquifer of Oligocene and Eocene age, the Beaufort aquifer of Paleocene age, and four Upper Cretaceous aquifers (the Peedee, Black Creek, and the Upper and Lower Cape Fear). For practical purposes, the surficial unit is not considered an "aquifer" since it cannot yield sufficient amounts of water even for domestic use. This limitation of its use is probably due to its small thickness (which limits available drawdown) near Camp Lejeune. The underlying hydrologic units are much thicker and are capable of yielding adequate supplies of water; therefore, the underlying units can be practically considered "aquifers" and are referred to as such in this report.

Each of the six aquifers mentioned above provide drinking water to many industries, municipalities, and private well owners throughout the eastern Carolinas and have been described in detail by many authors including Cardinell et al (1993), Trapp (1992), and Eimers et al (1990). The surficial unit and the Castle Hayne Aquifer were the only hydrologic units modeled in this effort because: 1) the contaminants beneath MCB, Camp Lejeune are either in the surficial unit or in the Castle Hayne Aquifer; 2) only the Castle Hayne Aquifer provides the drinking water for the Base; and 3) the underlying aquifers are over 400 feet deep and effectively isolated by the Beaufort confining unit. The other five aquifers were not modeled in this effort and are not discussed further here.

According to the data collected by Baker during site-specific RI studies, the surficial unit consists mainly of a fine sand with silt, although medium-grained sand occurs to a lesser extent. Across the Base, the thickness of the surficial unit ranges from 0 to 73 feet. These deposits are undifferentiated Pleistocene and recent sediments. Also, sand beds of the Belgrade Formation of Miocene age are considered part of the surficial unit (Cardinell et al, 1993). The bottom of the surficial unit is at or near mean sea level over most of the Base.

The Castle Hayne confining unit underlies the surficial unit and overlies the Castle Hayne Aquifer. It is comprised of clay and/or sandy clay from one or more of the following lithologic units: the lower portion of the Miocene Belgrade Formation, the upper portion of the Oligocene River Bend Formation, or the upper portion of the Eocene Castle Hayne Formation (Cardinell et al, 1993). The thickness of this confining unit averages about nine feet near Camp Lejeune and has been breached by the New River and some of its larger tributaries. This observation is one of the keys to understanding groundwater flow near the Base: the localized absence of the confining unit near the New River (or a large tributary) allows a strong hydraulic communication between the surficial unit and the Castle Hayne Aquifer. Cardinell et al (1993) graphically contoured the thickness of the Castle Hayne confining unit (Figure 3-1).

In contrast to the classification of lithologic units, the classification of hydrologic units or aquifers depends only on hydraulic conductivity. There must be a distinction between the two types of classification: the silty sand bed below the Castle Hayne confining clay may be considered lithologically as part of the Belgrade Formation, but hydrologically it belongs to the same hydrologic unit as the Castle Hayne Aquifer. The BRAGS conceptual model used only distinctions in hydraulic conductivity to define layers.

The Castle Hayne Aquifer lies beneath the Castle Hayne confining unit and consists of the lower portions of the Oligocene River Bend Formation and the Eocene Castle Hayne Limestone. In the vicinity of Camp Lejeune, the Castle Hayne Aquifer consists mainly of fine sand, shell rock and limestone. The upper portions of the aquifer consist of calcareous sand with discontinuous silt and clay beds (most likely the River Bend Formation). The calcareous sand becomes more limy with depth (Cardinell et al, 1993). At Site 73, two conspicuous layers of indurated (and subsequently fractured) fossiliferous limestone occur at elevations of approximately -30 to -50, and -80 to -100 feet referenced to mean sea level (msl) (see boring logs for Site 73 RI Report, 1997). The limestone layers may indicate the top of the Castle Hayne

Limestone and seem to be the most productive subunit of the Castle Hayne Aquifer as evidenced by the screened intervals of the Courthouse Bay supply wells (see Table 4-1). Harned et al (1989) constructed a typical figure of water supply wells at the base from data collected from water supply wells and USGS test wells; the figure shows a single 30-foot thick layer of limestone at elevations from -75 to -105 msl. While the thickness of the limestone layers may not be consistent, the presence of fossiliferous limestone seems to be laterally continuous across the Base.

In the vicinity of Camp Lejeune, the Castle Hayne confining unit and the upper Castle Hayne Aquifer have been incised by the meandering of the New River in ages past. Cardinell and others (1993) graphically contoured the top of the Castle Hayne Aquifer and a buried channel presumably created by the New River is evident in the southern half of Figure 3-2. This buried channel is significant in that it suggests that the Castle Hayne confining clay is breached near Courthouse Bay. This is, in fact, what was found in borings at Site 73, adjacent to Courthouse Bay (Baker, 1996b). This connection would provide hydraulic communication between the surficial unit and the Castle Hayne Aquifer and possibly allow uninhibited contaminant migration into the Castle Hayne Aquifer.

The bottom of the Castle Hayne dips to the east across the Base at an average gradient 0.004 feet/foot (ft/ft), (Cardinell et al, 1993).

3.3 Conceptual Model of Groundwater Flow

Wilder and others (1978) calculated an overall hydrologic budget for a typical location in the eastern Coastal Plain in North Carolina (see Figure 3-3): precipitation averages about 50 inches/year; five inches/year is lost to surface runoff; 34 inches/year is lost due to evaporation and plant transpiration. Total recharge to the water table is then about 11 inches/year. Of this amount, about 10 inches/year is discharged to surface water bodies as base stream flow. The remaining one inch/year leaks into the lower units (e.g., the Castle Hayne Aquifer and underlying units). Other estimates of regional recharge to the water table range from 12 to 20 inches/year (Geise et al, 1991) and also 15 to 22.5 inches/year (Leahy & Martin, 1993). However, even in studies where there were higher estimates of recharge to the water table, the estimates of vertical seepage to the underlying aquifers remained at 1 inch (Geise et al, 1991 and Eimers et al, 1994).

Precipitation falling on the upland areas of the eastern Coastal Plain generally moves vertically downward and generally flows horizontally toward the nearest groundwater discharge area: stream, river, bay, etc. (see Figure 3-4). As groundwater approaches the nearest discharge point (e.g. a stream or river), it may encounter a low hydraulic conductivity units (silt or clay) in which leakage through the layer is predominantly vertical. Near the discharge area, the head in the surficial water-bearing zones is reduced by the change in the surface relief at the surface water body; however, the pressure in the deeper aquifers remains higher than that in the surface water body. In the immediate vicinity of the discharge area, the particle responds to the vertical gradient in the deeper aquifers and moves vertically upward to the surface water body. The resulting flow path of a "typical" particle of groundwater in three-dimensions would therefore result in a curvilinear path from the recharge area to the discharge area.

Most of the precipitation falling in the middle of the Coastal Plain generally does not flow very far vertically, but flows horizontally to the nearest groundwater discharge area: stream, river, bay, etc. (see Figure 3-5); however, some of the precipitation infiltrating into the upland (recharge) areas (estimated at about 1"/year) manages to move downward toward the bottom of the unconsolidated sediments in response to the downward vertical head. At some depth, depending on the pressure head, groundwater stops migrating downward and starts to move horizontally toward the east.

As groundwater approaches the ocean, the pressure head in the upper aquifers is reduced by the change in the surface relief (ultimately to sea level). The pressure in the deeper aquifers beneath the coast remains higher than sea level and the vertical gradient in the deeper aquifers becomes vertically upward. The fresh groundwater then flows vertically toward the surface in the vicinity of the freshwater-saltwater interface near the ocean.

This freshwater-saltwater interface is the area where the saltwater from the ocean is at equal pressure with the freshwater from the land. As such it represents a no-flow boundary that is relatively stable in position (unless hydraulic stress such as pumping is introduced on either side of the interface). The fresh water then moves upward toward the coast and parallel to the interface. The pressure head in the aquifers is such that the groundwater is forced to the surface through the confining layers. The travel time for a pathway as described here (from the Fall Line to the shore) would be on the order of centuries or millennia. The rivers, lakes, and streams along the coast are the ultimate discharge points for fresh groundwater in the Atlantic Coastal Plain aquifers.

The natural groundwater discharge areas around Camp Lejeune are the New River and all of its tributaries (including swamps, wetlands, and streams) and the Atlantic Ocean. Most of these are at or very near mean sea level. Anthropogenic (man-made) discharges include a system of over 100 water supply wells in the Castle Hayne Aquifer at MCB, Camp Lejeune. In 1993, 68 of those wells pumped an average of almost 7 million gallons per day, according to information supplied by the Base Water Department. Some of the wells have been taken off-line and/or decommissioned because of high levels of organic contamination (e.g., HP-651), others due to poor well performance.

3.4 Hydraulic Characteristics

3.4.1 Surficial Unit

The hydraulic conductivity of the surficial unit has been measured by slug and pumping tests conducted during various RI and UST investigations. The average of the pumping and the slug testing in the surficial unit at IRP Sites 73 and 82 was 3.0 feet per day (ft/d). These data are presented in Table 3-1. The procedures and results of the shallow pumping test at Site 82 are discussed in Appendix A.

3.4.2 Castle Hayne Confining Unit

Between the surficial unit and the Castle Hayne Aquifer lies the Castle Hayne confining unit. Leakance of an aquitard (e.g., a clay and/or silt confining unit) is defined as the vertical hydraulic conductivity of that aquitard per foot of aquitard thickness (K_v/b). Leakance values for the Castle Hayne confining unit found by Trapp (1992) ranged from $1x10^{-6}$ to $1x10^{-4}$ ft/day/ft. Corresponding vertical hydraulic conductivity values for a 10 foot-thick unit range from $1x10^{-5}$ to $1x10^{-3}$ ft/d. At Site 73, the vertical hydraulic conductivity of the Castle Hayne confining clay unit was measured to be $2.6x10^{-7}$ cm/sec or $7.3x10^{-4}$ ft/d; the corresponding leakance value for a ten foot-thick clay unit would be $7.3x10^{-5}$ ft/day/ft which is within the stated range of Trapp.

3.4.3 Castle Hayne Aquifer

Several pumping tests were performed in deep wells in various locations around the Base: DRW-1 (Site 82 by Baker/OHM), supply well HP-642 (ES&E Inc.), supply well HP-708 (USGS), and test well X24s2x (NC DENR). The results of these tests indicated that the average hydraulic conductivity of the

Castle Hayne Aquifer is very similar to that of the surficial unit with values averaging 2.85 ft/d ($1x\ 10^{-3}$ cm/sec) and ranging from 0.09 ft/d (in the upper silty portions) to 8 ft/d ($7x10^{-4}$ to $3x10^{-3}$ cm/sec). The Castle Hayne hydraulic conductivity data from various sites and other hydrogeologic studies are summarized in Table 3-2. The previous studies by the USGS, NC DENR and ESE (Cardinell et al, 1993) resulted in values of hydraulic conductivity ranging from 2.3 ft/d to 4.9 ft/d, using values for saturated thickness of 308 to 382 feet.

These hydraulic conductivity values are indicative of fine sand and/or silty sand (Heath, 1983). In contrast, several USGS papers have been published that estimate the regional hydraulic conductivity of the Castle Hayne Aquifer in North Carolina as being one or more orders of magnitude greater than the site-specific values stated above (e.g., an estimated average of 65 ft/d, Winner & Coble, 1989). The highly permeable and relatively thin (10-20 feet thick) layers of indurated and fractured limestone within the Castle Hayne may be the reason for such high conductivity value estimates. When a highly permeable layer is tested via pumping (as the USGS did), the resulting transmissivity value is measured directly, independent of the unit's thickness. The calculation of the hydraulic conductivity value depends upon the interpretation of the thickness of the unit being tested. This may explain the apparent difference between the two sets of hydraulic conductivity data: a single transmissivity value divided by a large thickness (i.e., the entire thickness of the Castle Hayne Aquifer) would yield a lower hydraulic conductivity than for a thinner (limestone) layer. This modeling effort assumed an average thickness of 350 feet for the entire Castle Hayne Aquifer.

Another possible explanation for the difference between the regional and site-specific data could be the natural variations in hydraulic conductivity that can result from different depositional facies within the same chronostratigraphic unit, or perhaps post-depositional reworking by fluvial and/or tidal action. The large fraction of fine sand and silt in the upper portion of the Castle Hayne near MCB, Camp Lejeune indicates a relatively low to medium energy, shallow water environment of deposition.

SECTION 3.0 TABLES

TABLE 3-1
Hydraulic Conductivity Data
from IRP Sites 6, 82, and 73
Surficial Unit

BRAGS, CTO-0140
MCB, Camp Lejeune, North Carolina

Well	Hydraulic Conductivity (ft/day)	Test Method
SITE 73		
73-MW01	0.14	FH SLUG
73-MW01	0.18	RH SLUG
73-MW03	4.4	FH SLUG
73-MW03	4.4	RH SLUG
73-MW11	1.1	FH SLUG
73-MW11	1.0	RH SLUG
73-MW13	0.50	FH SLUG
73-MW13	0.35	RH SLUG
73-MW20	1.1	FH SLUG
73-MW20	1.1	RH SLUG
73-MW21	3.5	RH SLUG
73-MW22	1.8	FH SLUG
73-MW22	1.6	RH SLUG
73-MW23	3.6	RH SLUG
MW-15	5.1	PUMPING
MW-17	11.0	PUMPING
Dist/Draw	14.7	PUMPING
SITE 6/82		
SRW-1	1.47	PUMPING
SP-2	0.91	PUMPING
SP-1	1.31	PUMPING
6GW-34	2.5	PUMPING
SP-3	3.61	PUMPING
Minimum	0.14	
Maximum	14.7	
Average	3.0	
Standard Deviation	3.6	

TABLE 3-2
Hydraulic Conductivity Data from the Castle Hayne Aquifer

BRAGS, CTO-0140
MCB, Camp Lejeune, North Carolina

Well	Transmissivity (sq ft/day)	Thickness (ft)	Hydraulic Conductivity (ft/day)	Tested by
HP-708 lo	1140	382 (2)	3.0	USGS (1)
HP-708 hi	1325	382 (2)	3.5	USGS (1)
HP-642 lo	820	355 (2)	2.3	ESE, Inc. (1)
HP-642 av	1280	355 (2)	3.6	ESE, Inc. (1)
HP-642 hi	1740	355 (2)	4.9	ESE, Inc. (1)
X24s2x	900	308 (2)	2.9	NC DEHNR (1)
SITE 73				
73-MW01B	224	350	0.64	Baker - Slug (3)
73-MW01B	133	350	0.38	Baker - Slug (3)
73-MW11B	228	350	0.65	Baker - Slug (3)
73-MW11B	119	350	0.34	Baker - Slug (3)
73-MW15B	32	350	0.09	Baker - Slug (3)
73-MW15B	49	350	0.14	Baker - Slug (3)
SITE 6/82				
DRW-1	1081	350	3.09	Baker - Pumping (4)
6GW-1D	1856	350	5.30	Baker - Pumping (4)
DP-2	1179	350	3.37	Baker - Pumping (4)
DP-1	2928	350	8.37	Baker - Pumping (4)
6GW-15D	2054	350	5.87	Baker - Pumping (4)
Minimum	32	0	0.09	
Maximum	2928	350	8.37	
Average	1005	226	2.85	
Standard Deviation	828	172	2.35	

(1) - SOURCE: Cardinell et al, 1993, Table 4.
(2) - SOURCE: Cardinell et al, 1993, Table 3.
(3) - SOURCE: Baker Environmental, Inc., 1997b
(4) - SOURCE: Baker Environmental, Inc., 1996a

SECTION 3.0 FIGURES

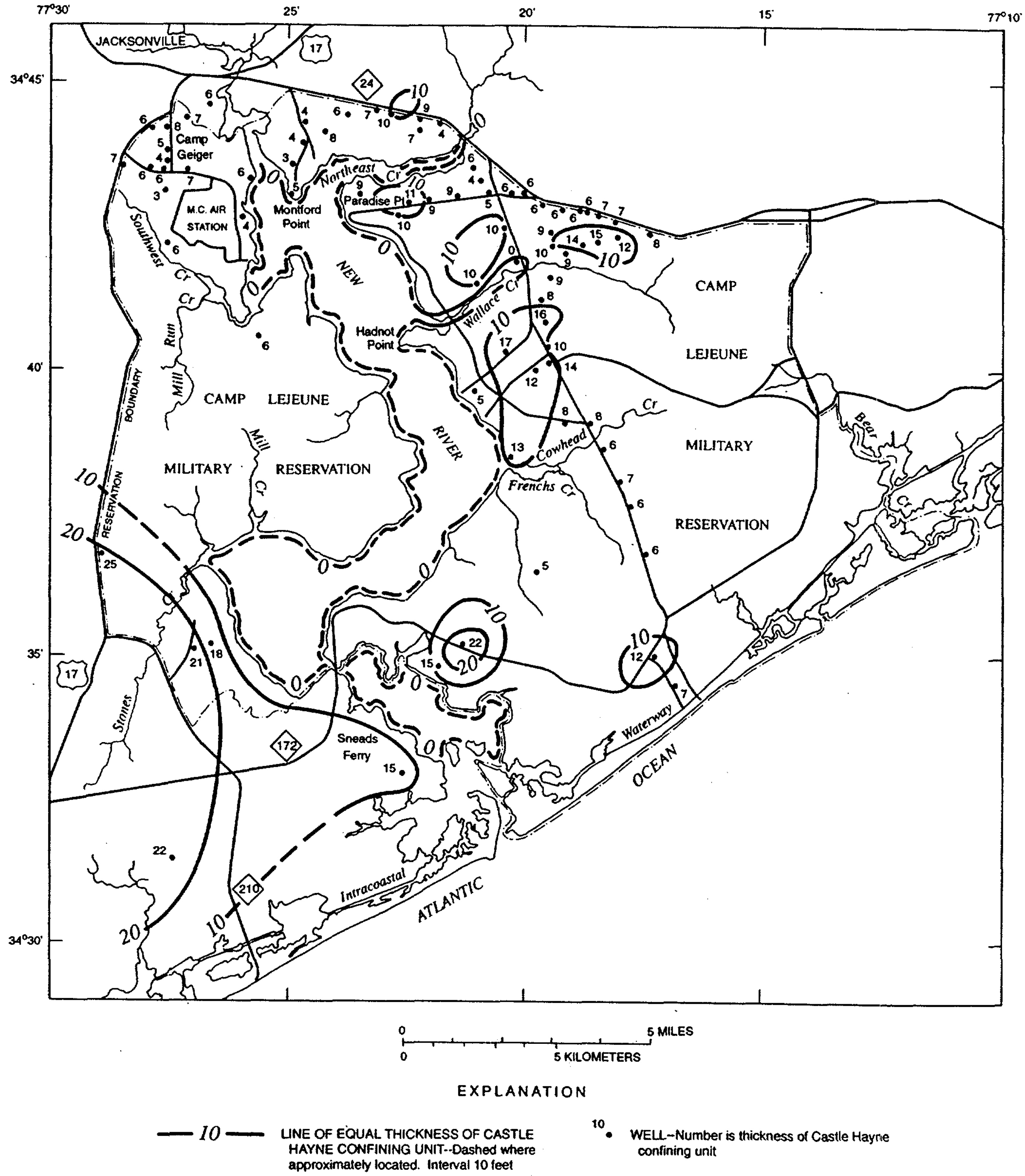


Figure 3-1 Isopach Contour Map -- Castle Hayne Confining Unit (taken from Cardinell et al, 1993)

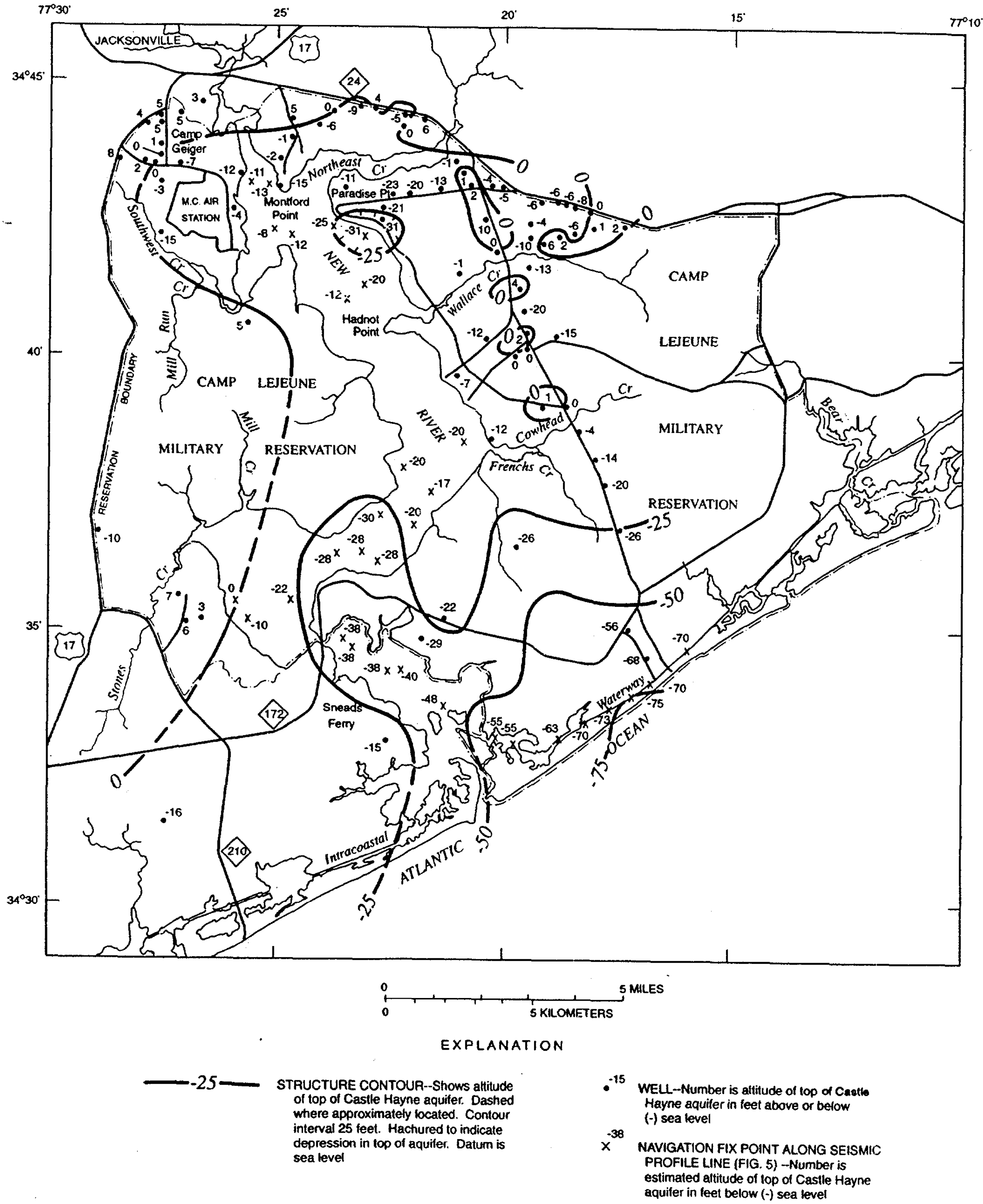


Figure 3-2 Elevation Contour Map -- Top of Castle Hayne (taken from Cardinell et al, 1993)

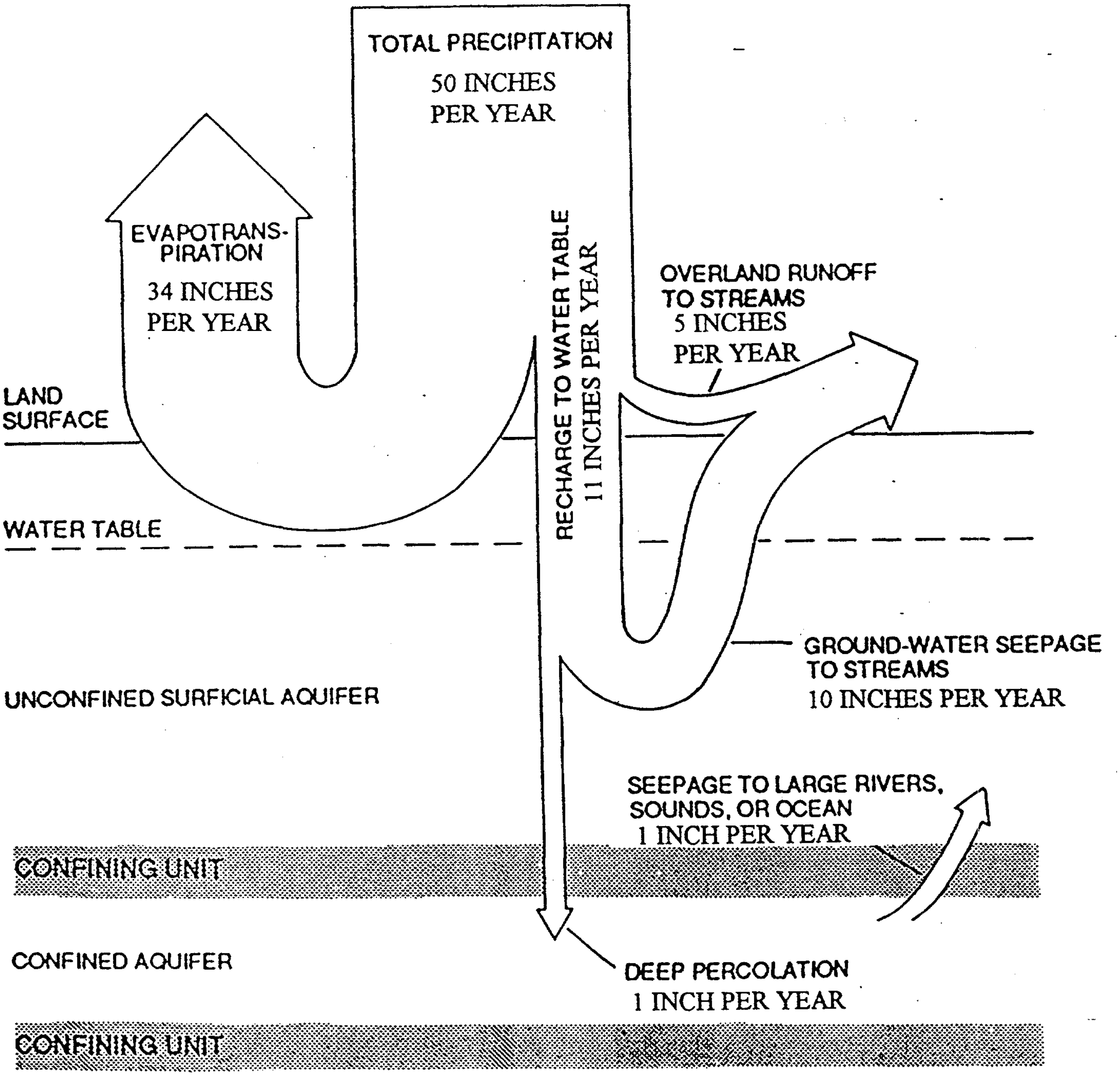


Figure 3-3 Typical Annual Water Budget (modified from Geise et al, 1991)

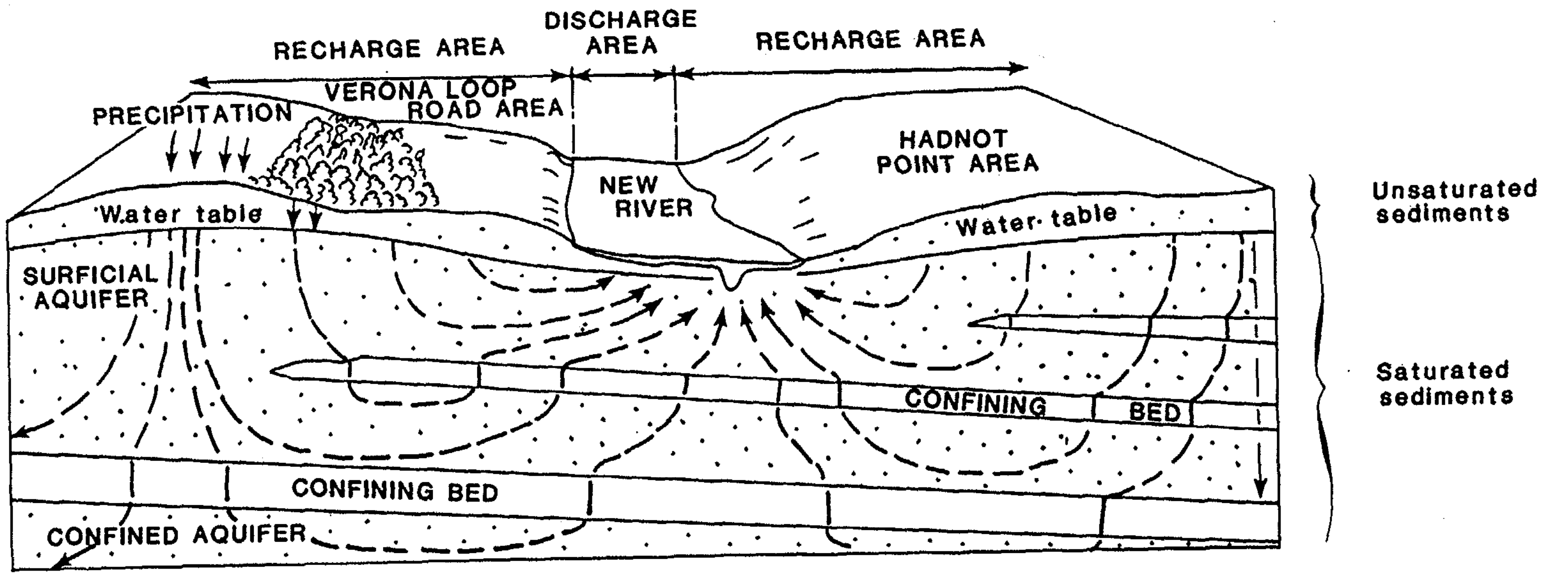


Figure 3-4 Idealized Hydrogeologic Cross-Section of New River (taken from Harned et al, 1989)

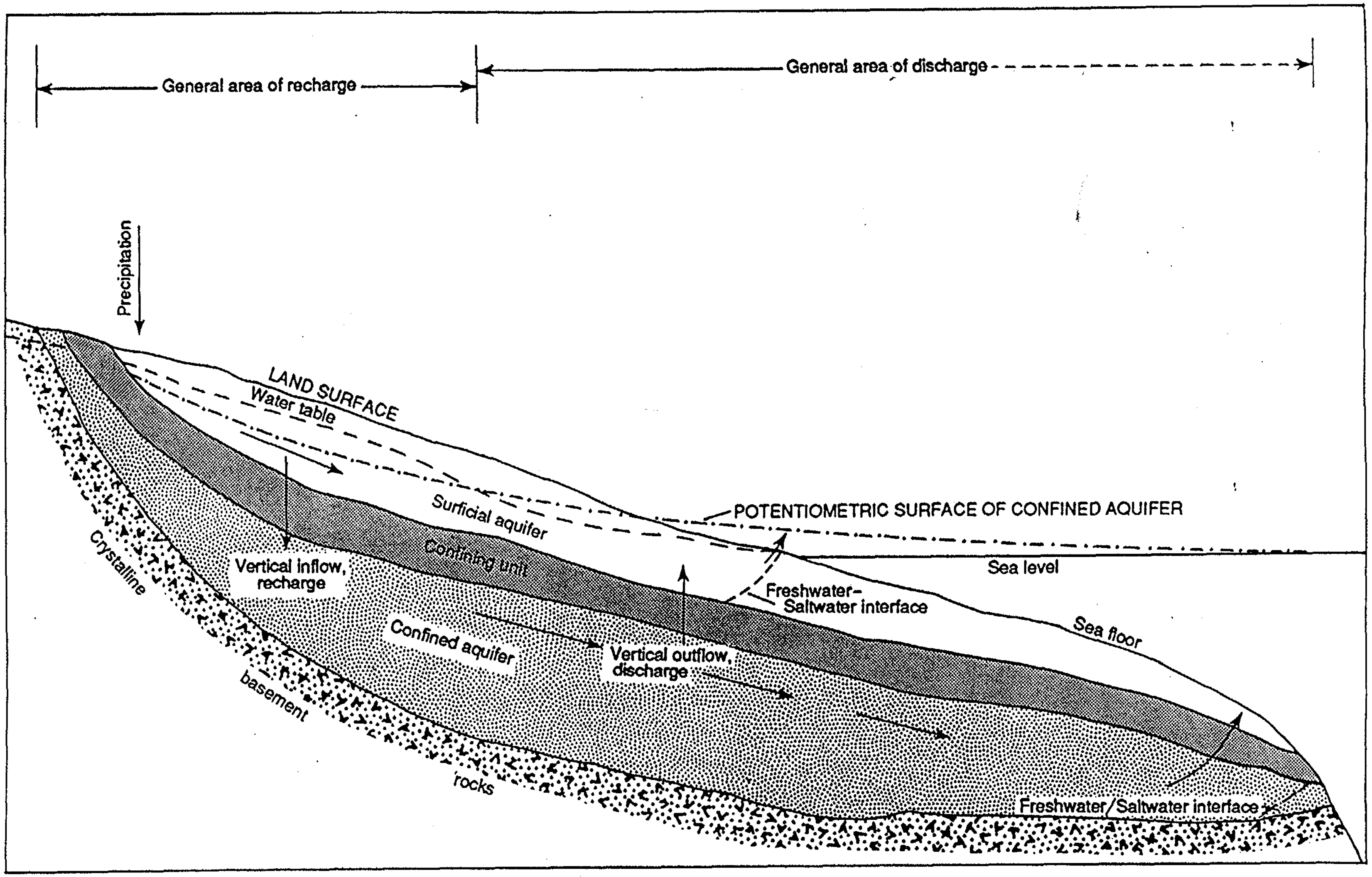


Figure 3-5 Idealized Hydrogeologic Profile of New River (taken from Geise et al, 1991)

4.0 BRAGS GROUNDWATER FLOW MODEL

The groundwater flow regime beneath MCB, Camp Lejeune was simulated by using the model code MODFLOW (McDonald & Harbaugh, 1988), a numerical groundwater flow code initially developed by the USGS and modified to run on IBM-compatible computers. This code was chosen because it has been extensively tested and documented in many applications and was appropriate for this complex, three-dimensional groundwater flow system.

The simplified governing (partial differential) equation used by the numerical model (MODFLOW) is:

$$\delta(K_{xx}\delta h/\delta x)/\delta x + \delta(K_{yy}\delta h/\delta y)/\delta y + \delta(K_{zz}\delta h/\delta z)/\delta z - W = S_s\delta h/\delta t$$

where:

- x, y, and z are Cartesian coordinates aligned with the major axes of hydraulic conductivity
- K_{ii} is the principle component of the hydraulic conductivity tensor in the i direction
- h is the potentiometric head or water table elevation
- W is a volumetric flux per unit volume of aquifer and represents sources and/or sinks of water
- S_s is the specific storage capacity of the porous material
- t is time

This equation describes the movement of water through a porous medium. For a steady-state model such as this, the right side of the equation becomes zero because the change in head with time is assumed to be zero. Together with the specification of initial and boundary conditions, this equation constitutes a mathematical model of groundwater flow.

MODFLOW can accommodate confined or unconfined conditions and uses input parameters of hydraulic conductivity, aquifer thickness, recharge, evapotranspiration, porosity, storativity, and specific yield to calculate water levels at various locations within the model boundaries. Each of the inputs can be varied spatially across the model grid so that by changing the parameters, a match to actual field conditions can be accomplished.

In order to use MODFLOW, it was necessary to discretize the domain into cells, each of which had a "node" containing the properties (e.g., hydraulic conductivity) and/or boundary conditions (e.g., rivers, wells) that approximated the conditions found at the site. For example, a well was reproduced by a "well" cell that specifies the flow into (recharge) or out of (discharge) the cell. Similarly, rivers, streams, swamps, and no flow boundaries were reproducible by one of the internal boundary cell types within MODFLOW.

Initially, the BRAGS model was to represent the seven aquifers beneath the Base: the surficial unit, the Castle Hayne Aquifer, the Beaufort aquifer, and four Upper Cretaceous aquifers (the Peedee, Black Creek, and the Upper and Lower Cape Fear); however, it was determined from the initial model runs that the aquifers below the Castle Hayne were not noticeably affected by changes to the top two layers. Layers representing the Beaufort aquifer and below were subsequently removed from the modeling process. In addition, little to no water level data from these units were available beneath the Base; therefore, the layers representing these aquifers could not be calibrated. This change improved the performance of the model and reduced the necessary memory requirements for its continual use. Electronic model input and output for the BRAGS model can be found on CD-ROM in Appendix B.

4.1 Finite-Difference Layered Grid

The finite-difference grid superimposed over the subject area has a uniform spacing: 1,000 feet by 1,000 feet square cells (see Figure 4-1). The grid has 101 rows (about 19 miles north to south) and 80 columns (about 15 miles east to west) over an area of approximately 285 square miles. The outer limits of the grid were chosen to be far enough away from the area(s) of pumping at the Base such that the boundaries of the grid would not interfere with any drawdown cones generated by pumping wells. Such interference would artificially increase or decrease the simulated drawdown, depending on the type of boundary being affected.

The fully-3D groundwater flow model consists of five layers (see Figure 4-2). From top to bottom they represent the surficial unit (layer 1), the Castle Hayne confining unit (layer 2), and three separate layers representing the upper (layer 3), middle (layer 4), and lower (layer 5) portions of the Castle Hayne Aquifer. The Castle Hayne Aquifer was divided into three portions because the "deep" monitoring well data represented only the upper portion of the Castle Hayne Aquifer. The water supply wells were generally screened in the middle of the Castle Hayne Aquifer where the fractured limestone occurs.

Therefore, the "deep" monitoring well target data was put into layer 3 and the supply well cells were put into the more permeable layer 4 (representing the limestone layer). Water elevation targets for the water supply wells were placed into layer 5 since the wells are also screened below the limestone.

4.2 Model Boundary Conditions

Boundaries in MODFLOW include external and internal boundaries. External boundaries can include specified (constant) head or general head boundary cells. General head boundary cells were used around the perimeter of the model to simulate the regional gradients of ambient groundwater flow. Internal boundaries include well, river, stream, and drain cells. For the BRAGS model, no stream cells were used. Internal boundaries were well, river and/or drain cells. Some (8%) of the cells in the model were inactive (no-flow) due to their location in the Atlantic Ocean.

Any cell in which the water elevation does not change appreciably over time such as the Atlantic Ocean were assigned specified (constant) head cells. There is no input of bottom elevations or conductances to this type of cell. The cell recharges or discharges to/from as much volume as the aquifer needs to keep the water elevation constant.

Any cell in which the elevation of the water and the bottom surface of the water body was used to simulate the surface water to groundwater interaction (recharge or discharge) which also had a relatively constant elevation such as the New River was simulated using river cells.

The remaining streams and swamps that were presumed only to receive groundwater runoff from the surficial unit were simulated by drain cells which are designed only to remove water from the groundwater system based on the elevation differences between the drain and the surrounding water table. Drain cells do not recharge groundwater.

4.2.1 Specified (Constant) Head Cells

The ocean was simulated by specified heads of zero (sea level) along the shore and no-flow cells further east. This arrangement presumes that all groundwater is discharging to the surface just west of (and along) the shoreline. It also presumes no east-west movement of the saltwater-freshwater interface along the shore.

4.2.2 General Head Boundary Cells

General head boundary cells are head-dependant flow cells that allow flow into or out of the cell depending on two things: 1) the head differential between the assigned value and that in the surrounding aquifer and, 2) an assigned constant of proportionality. In the BRAGS model, the assigned value of head represents a theoretical head value (e.g., of a surface water body) at some distance beyond the model boundary and the proportionality constant represents the hydraulic conductivity of the aquifer between the model boundary and the "theoretical" surface water body.

Four of the five layers had general head boundary cells placed along the outer boundaries to simulate the ambient groundwater gradient. The values of head assigned to each cell were chosen to represent the gradient of regional groundwater flow (0.000025 to the east in layer 1 and 0.0000125 to the east in the underlying layers, estimated from Geise et al, 1991). The proportionality constants were adjusted by trial and error during the calibration process until a reasonable fit was achieved at the boundaries.

4.2.3 Well Cells

Wells cells are specified (constant) flux boundaries which keep a constant flow rate throughout the specified time period. Positive values recharge to groundwater and negative values discharge from groundwater. MODFLOW assumes that each well fully penetrates the layer in which it is placed.

These cells were placed at the locations of the water supply wells and assigned negative (discharge) pumping rates in cubic feet per day. All available well locations were plotted even if they were turned off. This will help in the future if they are turned on again. The NC state planar coordinates (NAD 1983) of the water supply wells were converted from the latitude and longitude as recorded in Cardinell et al (1993).

The average pumping rates of the supply wells were calculated from 1993 total pumping data supplied by the MCB, Camp Lejeune, Base Water Department. Since the water supply wells are turned off at night, it was necessary to estimate the fraction of time the wells were pumped each day. This was done by taking the total gallons pumped from each well per year (P, gallons/year) and dividing it by 365 days/yr. This number was the average gallons pumped per day (p, gpd):

$$p = P/365$$

Then the maximum measured pumping rate (r, gallons/minute, or gpm) for each well was multiplied by 1,440 minutes per day to get the theoretical maximum daily rate (R, gpd) as if the well had been pumping day and night:

$$R = r \times 1{,}440$$

Next, p was divided by R to get the fraction of time the well was actually pumping per day (f, unitless):

$$f = p / R$$

For example, HP-603 pumped a total of 27,586,860 gallons in 1993. It's maximum pumping rate was 150 gpm (it has since been removed from service). The average daily rate was 27,586,860 gallons/365days = 75,580 gallons/day. The theoretical maximum that HP-603 could produce in one day was: 150 gpm x 1,440 min/day = 216,000 gallons/day. Assuming that when the well was on (pumping at its maximum rate), the fraction of time that the well was on-line was the ratio of average/maximum (75,580 gpd/216,000 gpd = 0.349) or 35% of the time. That would have been about 8 hours of pumping every day. This value varied from well to well. Table 4-1 presents the average daily pumping rates that were calculated for every well in gallons per minute and cubic feet per day.

4.2.4 River Cells

River cells are head-dependant flow cells in which the elevations of the surface water and river bottom are held constant (at surveyed or mapped elevations) and the thickness and conductance of the sediments control the flow rate of water to or from the cell. If the stream or pond level is higher than the surrounding groundwater, the river cell allows water to recharge the groundwater. Conversely, if the water level in the stream or pond is lower than the groundwater, the groundwater discharges to the surface water body.

The equation for river conductance C_{riv} is given by:

$$C_{riv} = (KLW)/M$$

where:

K = hydraulic conductivity of the river sediments;

L = length of the river in each cell;

W = width of the river in each cell; and,

M = thickness of the river sediments.

4.2.5 Drain Cells

Drain cells function similarly to river cells except that they cannot recharge the groundwater when the ambient water table drops below the drain elevation. Streams and swamps were represented by drain cells because it was reasonably assumed that they only receive groundwater discharge and were not recharging groundwater. In low-lying swamps and wetlands where the elevation of the water is lower than the surrounding water table, this assumption is reasonable as the wetlands would be receiving discharged water most of the year. However, in cases where there are wetlands atop hills where water is ponding above the water table, drain cells may not be the best representation; river cells may be better to provide a source of ponded water in this case.

4.3 Steady-State Modeling Process

In a steady-state groundwater flow model all values of drawdown are assumed to have reached equilibrium. That is, enough time is supposed to have passed with the wells pumping at constant rates that no additional drawdown is occurring. While rarely true in reality, this assumption can be considered valid when applied over the long term (years or decades) to understand how groundwater flows within the regime. The most important assumption of this approach is that the diurnal pumping schedule of the water supply wells has been averaged as if pumping were a continuous event.

In general, the extent to which the model assumptions match the actual subsurface conditions dictates the accuracy of any subsequent predictions. In order to get a realistic model, it was prudent to calibrate the model to match actual measured values of head. The "targets" of the calibration should be based on the statistics of the historical water level data where possible.

4.3.1 Calibration Targets

Water elevations measured at 21 IRP sites and one UST site around the Base during 1992 and 1993 provided "target" data for the BRAGS model. At these sites, the well locations were in such close proximity to each other that an average water level was calculated for use as the site target in the BRAGS groundwater flow model. This limited the number of targets to a manageable size (142 head targets in layers 1, 3, and 5).

From the average water levels in the shallow wells, 23 targets were established to which the surficial unit (layer 1) would be calibrated. In layer 3 (the upper portion of the Castle Hayne) there were 29 targets representing the "deep" and "intermediate" water level data. Layer 5 (the lower portion of the Castle Hayne Aquifer) contained 90 targets developed from the water level data from the supply wells. Because the water levels in the supply wells had been collected by the USGS over many years and in different ways, there was no way of knowing whether the water level data represented static conditions in these pumping wells at the time of collection. Therefore, more credence was given to the data in layers 1 and 3 than those in layer 5; therefore, the calibrations in layers 1 and 3 were deemed more accurate than that in the bottom layer.

4.3.2 Calibration Methods

The calibration process used the "trial and error" method in which the results of each run were examined statistically to determine the degree of "fit" of the results. No comprehensive groundwater contour maps exist for the Base; the only such maps were those pieced together by Harned et al (1989) and those were spaced rather far apart. Statistics used were mean error (ME), mean absolute error (MAE), standard deviation of the errors (SDE), and the root mean square error (RMSE). After each run, one or more input values (e.g., horizontal and/or vertical hydraulic conductivity) and/or their spatial distributions were changed and the model rerun. Changes were made to various parameters in those areas of the grid where the error between simulated and measured water levels was large. In this "trial and error" method, not all changes were for the better, some had to be changed many times to find a "better" value or distribution. This process continued until a reasonable fit was achieved.

The definition of "reasonable fit" is relative and depends upon many things including the amount of fluctuation in naturally-occurring water levels and upon the reliability of data collection methods. For the Marine Corps Air Station (MCAS), Cherry Point groundwater flow model (Eimers et al, 1994), statistics were also used to judge the adequacy of "fit." As discussed below, this effort had comparable statistics to those used at MCAS, Cherry Point.

4.3.3 Statistical Evaluation of Calibration

The difference between a measured value (or target) and a simulated value is called an error. The average of the all the errors should be close to zero for an accurate model because that indicates that the errors higher than the targets are balanced by the errors below the targets. A highly positive or negative mean error (ME) would indicate an inaccurate model in which the water levels are all too low or too high, respectively. The ME is a good indication of accuracy but not of precision (or data dispersion). As long as the errors were balanced a model can be considered accurate, but the fit is better measured by the more useful indicators of precision. These include the mean absolute error, MAE, the standard deviation of the errors, SDE, and the root mean square error, RMSE. These statistical values are better indicators of fit than the ME alone because smaller values indicate less dispersion from the targets and that the simulated values collectively match the targets more closely (see introductory statistical reference such as Dixon & Massey, 1983).

4.4 Calibrated Results of Simulation

This section describes the inputs and outputs of the calibrated simulation. Unless otherwise indicated, the values used in the model were taken directly from the values discussed in the previous sections.

Table 4-2 presents the errors of each target in the BRAGS model. The end of the table shows the statistics for each layer as well as for the entire model. The final ME value for all three layers in the BRAGS model was -0.85 feet, which indicates that the average of all the simulated water levels in all three layers was 0.85 feet lower than the measured water levels. The MAE for all three layers was 4.46 feet. The SDE was 6.10 feet, and the RMSE was 6.14 for the final calibration.

The comparable statistics between Camp Lejeune (three targeted layers) and MCAS, Cherry Point (six targeted layers) are as follows:

		MCAS, Cherry Point	MCB, Camp Lejeune
		(Eimers et al, 1994)	(Baker,currentreport)
Mean Error (ME)	=	0.20 feet	-0.85 feet
Mean Absolute Error (MAE)	=	4.35 feet	4.46 feet
Standard Deviation of Errors (SDE)	=	5.70 feet	6.10 feet
Root Mean Square Error (RMSE)	=	5.70 feet	6.14 feet
Range of Errors	=	± 17 feet	± 23 feet
Range of Observed Elevations (R)	=	45 [-8 to 37 feet msl]	43 [-5 to 38 feet msl]
SDE/R	=	13%	14%

These statistical values of the two models compare favorably; the MAE, RMS, and SDE values indicate that the simulated values at Camp Lejeune are as close to the observed targets as are those at MCAS, Cherry Point. The ratio of SDE to R can be used as an indication of adequate calibration. The ratios suggest that the two models are calibrated to an equivalent level.

The range of errors may seem rather large for both models; however, for the Camp Lejeune BRAGS model, the largest errors occurred at locations having less than completely reliable measured data (i.e., in water supply wells with limited and sometimes dubious available data). With one exception, all of the errors in the surficial unit and upper Castle Hayne Aquifer (layers 1 and 3) are within ±10 feet of their targets; most of the error in layers 1 and 3 is associated with one data location (Site 69). The significance of this will be discussed below. In layer 5, all the modeled heads are within ±20 feet of the targets.

Figure 4-3 shows the graph of modeled head values as a function of the observed target head values. With one exception (Site 69), the heads in the three targeted layers (1, 3, and 5) are within ±10 feet of the established targets. Figure 4-4 shows this more clearly: the target data point at Site 69 was very close to the New River but was also elevated more than 25 feet above mean sea level (msl). If the data from Site 69 were excluded as targets, the statistics would show that the current groundwater flow model predicts the rest of the data with much better accuracy than the aforementioned numbers indicate: ME = -0.28; MAE = 2.79; and RMSE = 3.78.

Figures 4-5 and 4-6 show the spatial distribution of the errors in layer 3. Again, with one exception, the heads are within ±10 feet of the targets. The anomalous value is from Site 69 without which the statistics are much improved: ME = -0.94; MAE = 1.85; and RMSE = 2.57.

Figure 4-7 and 4-8 show that the modeled heads for the lower Castle Hayne Aquifer are within ±20 feet of their targets. The error for layer 5 is larger than that in layers 1 and 3 due to several factors: 1) water level data from the water supply wells may have been measured before the wells had fully recovered from pumping, 2) even if the supply wells were allowed to fully recover internally before measurement, the influence of nearby active wells would not have allowed truly non-pumping conditions in many wells, 3) the wellheads of the supply wells had not been surveyed for vertical nor for horizontal control; elevations were estimated from topographic maps.

Figures 4-9 and 4-10 show the spatial distribution of error in layer 1. Figure 4-11 and 4-12 show the spatial distribution of error in layer 3. Figures 4-13 through 4-16 show the spatial distribution of error in layer 5. These figures show error bars for each data station: the bars represent ± one standard deviation from the mean (where the data were available); when data were insufficient to produce a standard deviation, a confidence interval of no more than five feet was chosen, depending upon the type of data point.

4.4.1 Layer 1 -- Surficial Unit

4.4.1.1 Input

A uniform value of recharge of 11 inches per year was used in this model. This value was estimated based on several USGS studies (see section 3.3 for discussion) and was also calibrated to site-specific hydraulic conductivity data. Recharge occurred only in layer 1. Layer 1 was unconfined and bottom elevations in layer 1 ranged from -70 feet to +10 feet msl (see Figure 4-17).

The general head boundary (GHB) cells were set to about 50 feet above mean sea level (msl) in the inland areas around the Base (see Figure 4-18). The conductance of the GHB cells (C_{ghb}) was set at 2,000 ft^2/d. The Atlantic Ocean was represented by a specified (constant) head value of +0 feet msl along the shoreline.

River cells were used to represent the New River and its elevation was assumed to be mean sea level (see Figure 4-18). Using the input value of 5,000 ft^2/d for river cell conductance (C_{riv}) is reasonable assuming the following parameter values for the New River:

L = 1000 feet (average length of river cell in the New River)

W = 1000 feet (average width of river cell in the New River)

$M = 2$ feet (estimated thickness of river sediments)
$K = 0.01$ ft/d (or 3.5×10^{-6} cm/sec)

This K value is typical of a silty clay sediment which is reasonable for the bottom of the New River.

The elevations of the drain cells were the approximate elevations of the streams as determined by topographic mapping of the area (see Figure 4-18). Drain cells for streams were assigned a uniform conductance value (C_{drn} =5,000 ft^2/d). This translates to the following values:

$L = 1000$ feet (average length of drain cell along streams)
$W = 10$ feet (estimated average width of streams)
$M = 1$ feet (estimated thickness of stream sediments)
$K = 0.5$ ft/d (or 2×10^{-4} cm/sec)

This K value is typical of silt which would be expected in low energy streams.

Drain cells were also used to simulate three areas of upland swamps by assigning mapped elevations and a C_{drn} value of 500 ft^2/d. This yields the following value:

$L = 1000$ feet (average length of drain cell in wetlands)
$W = 1000$ feet (average width of drain cell in wetlands)
$M = 5$ feet (estimated thickness of confining clay)
$K = 2.5 \times 10^{-3}$ ft/d (or 9×10^{-7} cm/sec)

This K value is typical of the vertical hydraulic conductivity of a clay which would underlie a swampy area.

Horizontal hydraulic conductivity K_h in layer 1 was uniform at 5 ft/d. This was the calibrated value adjusted from the average hydraulic conductivity value of 3 ft/d (from shallow pumping and slug tests at Sites 73 and 82).

Vertical hydraulic conductivity was assumed to be 0.1 times the value of horizontal hydraulic conductivity in the surficial unit (K_z=0.5 ft/d in layer 1). This assumption was based on the relatively large calculated average value of vertical hydraulic conductivity (1.7 ft/d) by the Neuman method at Site 82 (see Appendix

A). This value may not be indicative of the entire Base as the confining unit is absent at Site 82. Vertical anisotropy is often unknown and is estimated during calibration. Vertical anisotropy ratios ranging from 1 to 1,000 are common in model application (Anderson & Woessner, 1982).

No well cells were used in the surficial layer of the BRAGS model.

4.4.1.2 Output

Figure 4-19 shows the water table contours across MCB, Camp Lejeune. The map shows that the New River and its tributaries are the main areas of groundwater discharge from the surficial hydrostratigraphic unit. Between the streams (localized discharge areas) are recharge areas; this was the expected pattern of flow in the surficial unit, based on the conceptual model.

With one exception, the simulated water levels in layer 1 were within 8 feet of their targets. Table 4-2 shows that the ME in layer 1 was -1.08 feet, the MAE was 3.48 feet, the SDE was 5.38 feet, the RMSE was 5.37. As indicated by the value of the SDE for layer 1, about 66% of the simulated heads were within 5.4 feet (one standard deviation) of their targets and 95% of them were within 11 feet (2 standard deviations). These compare favorably to: MAE = 3.87, SDE = 5.0, and RMSE = 5.0 for the surficial unit in the MCAS, Cherry Point groundwater model (Eimers et al, 1994).

Table 4-3 is a summary of the simulated hydrologic budget for the BRAGS model. It shows that, of 11 inches of recharge per year, about 1.7 inches infiltrated into the Castle Hayne Aquifer. This value, while larger that the estimate by Wilder et al (1978) of 1.0 inches per year, is still relatively close to the estimate. The BRAGS value of deep infiltration may be larger due to the effects of supply well pumping which accounts for 0.6 inches per year on average (5,366 gpm). The cessation of pumping would tend to make the value of deep infiltration about 1.1 inches per year.

4.4.2 Layer 2 - Castle Hayne Confining Unit

Layer 2, representing the Castle Hayne confining unit, had a uniform thickness of 10 feet but varied in depth across the entire Base. The top elevation of layer 2 is the same as the bottom of layer 1. The bottom of the confining unit varied from +0 to -80 feet msl. Layer 2 comprised three values of horizontal hydraulic conductivity: 0.1 ft/d over most of the Base area, 0.00073 ft/d in selected places and 5 ft/d where the clay unit was breached. Vertical hydraulic conductivities was assumed to be 0.1 times the horizontal values. Figure 4-20 shows the leakance values used in layer 2 (where leakance = K_v/thickness = K_v/10 feet).

No GHB, well, river, or drain cells were used in layer 2 of the BRAGS model for MCB, Camp Lejeune because it is assumed that the flow in layer 2 is mostly vertical and no groundwater flows laterally through the boundaries of layer 2.

No river cells were needed in layer 2 to simulate leakage to the New River because the vertical permeabilities in layer 2 were used to simulate the presence or absence of the clay unit. Where no confining unit was present, the vertical hydraulic conductivity of layer 2 was much higher than in areas where a confining unit was indicated. The higher the vertical hydraulic conductivity, the more hydraulic "communication" between vertically adjacent units. The direction of vertical flow depends on the head differences in the adjacent units. The higher heads in the upper Castle Hayne (layer 3) provided the impetus for the upward leakage to the New River in layer 1.

4.4.3 Layer 3 - Upper Castle Hayne Aquifer

4.4.3.1 Input

Top elevations in layer 3 were identical to the bottom elevations of layer 2. Bottom elevations in layer 3 range from -40 feet to -130 feet msl (see Figure 4-21). The bottom of layer 3 is sloping from west to east across the study area. The GHB cells in layer 3 are set so that a very slight regional gradient (0.0000125, estimated from Geise et al, 1991) is flowing to the east (i.e., 25 feet msl at the western boundary and 24 feet msl at the eastern boundary). The uniform hydraulic conductivity value in layer 3 is 7 ft/d. This is a calibrated adjustment to the average from the pumping tests in the upper Castle Hayne Aquifer (3 ft/d). Vertical hydraulic conductivity was assumed to be 0.1 times the horizontal value in layer 3 (0.7 ft/d).

No well, river, or drain cells were used in layer 3 of the BRAGS model for MCB, Camp Lejeune.

4.4.3.2 Output

Figure 4-22 shows the piezometric surface contours in the upper Castle Hayne (layer 3) across MCB, Camp Lejeune. The map shows that the New River and its tributaries are still the main areas of groundwater discharge from the upper Castle Hayne Aquifer. The drawdown from the pumping wells is shown especially near Hadnot Point where the resulting groundwater elevation is near sea level.

All of the simulated water levels in layer 2 (with the exception of Site 69) were within 8 feet of their targets. Table 4-2 shows that the ME in layer 2 was -1.68 feet, the MAE was 2.57 feet, the SDE was 4.68 feet, and the RMSE was 4.89 feet. As indicated by the value of the SDE for layer 2, about 66% of the simulated heads were within 4.7 feet (one standard deviation) of their targets and 95% of them were within 9.4 feet (2 standard deviations). This match is very similar to that of layer 1. These compare favorably to: MAE = 4.89, SDE = 5.50, and RMSE = 6.30 for the upper Castle Hayne Aquifer in the MCAS, Cherry Point groundwater model (Eimers et al, 1994).

4.4.4 Layer 4 - Castle Hayne Fractured Limestone Unit

Layer 4, representing the highly conductive Castle Hayne fractured limestone, had a uniform thickness of 10 feet but varied in depth across the entire Base. The top elevations of layer 4 were the same as the bottom elevations of layer 3. The bottom of the confining unit varied from -50 feet to -140 feet msl. Layer 4 had a uniform horizontal hydraulic conductivity of 100 ft/d over the Base area. Vertical hydraulic conductivity was assumed to be 0.1 times the horizontal value (10 ft/d). This value was estimated assuming that a higher hydraulic conductivity unit existed in the lower portions of the Castle Hayne Aquifer. No direct hydraulic conductivity data exist that can confirm this, but it is consistent with the well logs around the Base and with regional values of hydraulic conductivity in the Castle Hayne Aquifer (Geise et al, 1991; Cardinell et al, 1993; Harned et al, 1989).

The water supply wells were placed into layer 4 of the BRAGS model for two reasons: 1) the high yields of these wells suggest a high conductivity layer and, 2) the well logs for most of the supply wells indicate that they are screened in one or more fractured limestone layers. Figure 4-23 shows the locations of the water supply wells around Camp Lejeune. Well cells were installed for all identified well locations around the Base but only those actually pumping in 1993 were given values of discharge as shown in Table 4-1. Discharge rate values for MODFLOW are in negative cubic feet per day.

GHB cells were used in layer 4 as boundaries and were identical to those of layers 3 and 5. No river or drain cells were used in layer 4 of the BRAGS model for MCB, Camp Lejeune.

4.4.5 Layer 5 -- Lower Castle Hayne Aquifer

4.4.5.1 Input

The top elevations of layer 5 are identical to the bottom elevations of layer 4. Bottom elevations in layer 5 range from -180 feet to -480 feet msl (Figure 4-24). The bottom of layer 5 is sloping from west to east across the study area.

The GHB cell boundaries in layer 5 are set the same as in layers 3 and 4. The regional gradient is flowing to the east. No river or drain cells were used in layer 5.

The uniform hydraulic conductivity value in layer 5 is 10 ft/d. Leakance is inactive in layer 5 as it is the bottom of the model. This has the same effect as a no-flow boundary at the bottom.

4.4.5.2 Output

Figure 4-25 shows the piezometric surface contours in the lower Castle Hayne (layer 5) across MCB, Camp Lejeune. As in layer 3, the map shows that the New River and its tributaries are the main areas of groundwater discharge from the lower Castle Hayne Aquifer. The drawdown from the pumping wells is shown especially along Brewster Boulevard (near Paradise Point) where the resulting groundwater elevation is near sea level.

Table 4-2 shows that in layer 5, all of the errors were less than 20 feet. Table 4-2 shows that the ME in layer 5 was -0.52 feet, the MAE was 5.33 feet, the SDE was 6.68 feet, and the RMSE was 6.66 feet. As indicated by the value of the SDE for layer 2, about 66% of the simulated heads were within 6.7 feet (one standard deviation) of their targets and 95% of them were within 13.4 feet (2 standard deviations). These compare favorably to: MAE = 5.45, SDE = 6.50, and RMSE = 6.50 for the lower Castle Hayne Aquifer in the MCAS, Cherry Point groundwater model (Eimers et al, 1994).

4.4.6 Three-Dimensional Analysis of Groundwater Flow

MODFLOW was used in the BRAGS model to generate 3-D flow vectors (directions and relative velocity at discrete points) in a map view and in an west-to-east cross-section, respectively. The length and size of each arrow represents its relative velocity compared to the other arrows. Figure 4-26 shows the map view of the northern portion of the Base in layer 4 (limestone) where the pumping wells are concentrated. The combined effects of the supply wells can be seen along Brewster Boulevard near Paradise Point where the flow directions have been reversed from the river back toward the wells. In Camp Geiger the effect is less but is still noticeable as the velocity of the ambient flow (from west to east upgradient of the wells) has been slowed considerably.

As shown on Figure 4-26, the controlling factors in determining where groundwater flows are the streams: they provide a differential head that allows deep groundwater to migrate upwards because the heads in the underlying aquifer are much greater than those in the streams. In combination with breaches in the confining clay unit, a vertical "escape route" is provided for deep groundwater. That is why the flow vector arrows are so large in upstream portions of each stream; the head differential is typically much larger there than near the New River. Also noteworthy are the larger flow vectors near the edges and the smaller flow vectors toward the middle of surface water bodies; this is also the result of the changing head differentials beneath the surface water.

Figure 4-27 shows an east-west cross-section between Verona Loop Road and Hadnot Point. It shows that groundwater flows like that envisioned in the conceptual model: downward in the upland recharge areas and upwards to discharge into local streams or the New River (see also Figure 3-4). The flow in the surficial unit is mostly vertical and the flow in the lower Castle Hayne (layer 5) is mostly horizontal. Again, the flow vectors are largest where the head differential is largest, in upstream portions of the tributaries to the New River. The flow vectors discharging to the New River itself are much smaller in magnitude because the head differential is smaller there.

Water supply wells intercept some of this water on its way to the New River but the individual effects are localized around the area near each well. However, where the supply wells are grouped together and their drawdown "cones" overlap are where the resulting groundwater elevations are lowest. Figures 4-22 and 4-25 show such an area along Brewster Boulevard (near Paradise Point) where the model predicts steady-state groundwater elevations in the upper and lower Castle Hayne Aquifer at or below sea level. These are the "danger zones" for saltwater intrusion into the Castle Hayne Aquifer. In order to mitigate this situation, pumping wells should be spread out as much as possible to avoid the creation of such zones and to preserve the potable groundwater quality of the Castle Hayne Aquifer.

4.5 Sensitivity Analysis

A sensitivity analysis was performed on the BRAGS model. Selected parameters were changed by ± 20 and 50% and the resulting effects were quantified statistically using the values of ME, MAE, and RMSE. The following six parameters were used in the sensitivity analysis: recharge, hydraulic conductivity (horizontal), leakance, general head boundary cell conductance, river cell conductance, and drain cell conductance.

4.5.1 Effects of Altering Recharge

Figure 4-28 shows the effect of changing the value of recharge from the calibrated input value (11 inches/year). The model is very sensitive to recharge, but no significant improvements were found based on changes to recharge input values. While a 15% increase in recharge would make the ME closer to zero, the values of RMSE and MAE would increase slightly.

4.5.2 Effects of Altering Horizontal Hydraulic Conductivity

Figure 4-29 shows the effect of changes to horizontal hydraulic conductivity, K. During the analysis, values of K were changed in every layer at once. While decreases in the K values reduced the value of ME, they produced unacceptable increases in RMSE and MAE.

4.5.3 Effects of Altering Leakance

Figure 4-30 shows the effect of changes to leakance, which is vertical hydraulic conductivity, K_v, divided by the unit thickness, b. During the analysis, values of leakance were changed in every layer at once. Decreases in leakance values reduced the value of ME toward zero and slightly improved the values of RMSE and MAE. It is possible that a reduction in the values of leakance may slightly improve the model.

4.5.4 Effects of Altering GHB Cell Conductance

The values of ME, RMS and MAE were not noticeably affected by changes in the conductance of the GHB cells (see Figure 4-31).

4.5.5 Effects of Altering River Cell Conductance

The values of ME, RMSE and MAE were slightly improved by reductions in the conductance of the river cells, but the changes were not significant (see Figure 4-32).

4.5.6 Effects of Altering Drain Cell Conductance

Figure 4-33 shows that the value of ME was slightly improved by decreasing the conductance of the drain cells by 50%. The values of RMSE and MAE were also slightly reduced. Therefore, a change in the conductance of drain cells may be recommended, depending upon the effects of other recommended changes.

4.5.7 Recommended Changes to the Model

Figures 4-34 through 4-36 show the relative sensitivity of the model to the six parameters. Figure 4-34 shows the response of ME to the changes: in descending order, the ME is most sensitive to recharge, hydraulic conductivity, leakance, drain conductance, and river conductance. The ME was not sensitive to the changes in GHB cell conductance. The ME is most sensitive to changes in recharge; although improvements were obtained by increasing recharge by about 15%, such a change is not recommended because it did not improve the other statistics (RMSE and MAE). Reductions of hydraulic conductivity in all five layers by 50% produced improvements in the value of ME; again however, such a change is not recommended because it did not improve the other statistics (RMSE and MAE). Reductions in drain cell conductances and leakance (50%) seemed to improve the ME. A 50% reduction of river cell conductance also slightly improved the ME.

Figure 4-35 shows that a 50% decrease in the values of leakance, drain cell conductance, and river cell conductance would improve the RMSE. On this figure it is shown that a 20% increase in K would slightly improve the RMSE. No improvements were noted by changing the calibrated value of recharge. The RMSE was not sensitive to changes in GHB cell conductance.

Figure 4-36 shows that no significant improvements to the value of MAE are obtained by changing the six calibrated input values.

In summary, the model is most sensitive to (in decreasing order):

1. Recharge
2. Hydraulic Conductivity (horizontal)
3. Leakance
4. Drain Cell Conductance
5. River Cell Conductance

The model was not sensitive to changes made in the conductance of the general head boundary (GHB) cells.

Recommended changes are to decrease leakance, drain cell conductance and river cell conductance values by at least 50% to lower RMSE and MAE values and to achieve an ME value close to zero. Changes to recharge, hydraulic conductivity, and GHB cell conductance are not recommended.

4.6 BRAGS Groundwater Flow Model Summary

The BRAGS groundwater flow model presented herein portrays the three-dimensional pattern of groundwater flow within the surficial units and the Castle Hayne Aquifer. It achieves the first two of the three objectives described in Section 1.1:

- Based on the conceptual model described in Section 2.0, the model describes how groundwater flows in three dimensions beneath the entire Base as well as under individual sites (Objective 1). The model reasonably predicts the elevation and flow direction of the groundwater in many areas around the Base where no data currently exist.

- The model demonstrates the effects of groundwater supply withdrawals on the surficial unit and the Castle Hayne Aquifer (Objective 2). The model demonstrates that discharge to the New River and its tributaries is the controlling factor on groundwater flow directions in the Castle Hayne Aquifer in the vicinity of Camp Lejeune. The model output indicates that the relatively high-volume withdrawal rates of the supply wells have a localized effect on the water levels in the Castle Hayne; however, large numbers of actively pumping wells in small areas have the potential to induce saltwater intrusion into the upper Castle Hayne Aquifer. This effect is most pronounced in Paradise Point along Brewster Boulevard.

Actively pumping water supply wells should not be grouped together in small areas but should be spread out in a line perpendicular to the ambient flow direction (not parallel to it) to avoid this situation.

- Although the BRAGS model does not directly address the third objective (i.e., predicting the relative effectiveness of various site-specific remediation schemes), it strongly indicated that the low volumes of water withdrawn from the surficial unit and/or the Castle Hayne Aquifer during such remedial actions will not seriously impact the water supply at the Base.

SECTION 4.0 TABLES

TABLE 4-1
Pumping Rates of Water Supply Wells at MCB, Camp Lejeune

Well Name* or Number	Well Depth (feet)	Screen Depth Top (feet)	Screen Depth Bottom (feet)	Casing Depth (feet)	Well Diameter (inches)	Average Daily Withdrawal in 1993 (MGD)	Regular or Emergency	TOTAL GALLONS* (1993)	MAX PUMPING RATE (GPM)	AVG PUMPING RATE (GPM)	% TIME ON
603	195	70	195	70	8	0.076	REGULAR	27,586,860	150	53	35%
606	210	80	210	80	8	0	REGULAR	0	345	0	0%
607	210	UNK	UNK	50	8	0.172	REGULAR	62,778,180	293	119	41%
609	145	65	145	65	8	0.097	REGULAR	35,546,040	162	67	42%
613	150	60	150	60	8	0.096	REGULAR	35,215,560	200	67	33%
616	147	95	147	95	8	0.124	REGULAR	45,406,200	200	86	43%
620	52	UNK	UNK	46	18	0.084	REGULAR	30,800,700	160	58	36%
622	227	UNK	UNK	50	UNK	0.245	REGULAR	89,503,200	310	170	55%
623	197	UNK	UNK	50	UNK	0.119	REGULAR	43,570,800	300	83	28%
628	200	60	145	50	8	0.065	REGULAR	23,663,640	150	45	30%
629	230	60	230	50	8	0.088	REGULAR	32,121,000	150	61	41%
632	200	UNK	UNK	63	UNK	0.105	REGULAR	38,260,800	240	73	30%
633	205	55	205	55	8	0.156	REGULAR	57,076,800	250	108	43%
635	215	65	215	65	8	0.097	REGULAR	35,391,300	200	67	34%
640	179	64	176	64	8	0.149	REGULAR	54,338,880	214	103	48%
641	178	108	168	52	8	0.177	REGULAR	64,624,380	281	123	44%
642	210	112	196	40	8	0	REGULAR	0	156	0	0%
652	183	120	178	50	10	0.044	REGULAR	15,978,240	146	31	21%
654	183	UNK	UNK	50	UNK	0.079	REGULAR	28,817,040	119	55	46%
661	135	50	135	50	10	0.2	REGULAR	72,839,820	269	139	52%
662	230	UNK	UNK	50	UNK	0.064	REGULAR	23,450,520	146	44	30%
663	180	130	180	50	10	0.151	REGULAR	54,986,400	210	105	50%
709	140	70	140	50	10	0.123	REGULAR	44,737,200	172	85	50%
710	140	70	140	50	10	0.074	REGULAR	26,938,800	105	51	49%
711	150	60	150	50	10	0.063	REGULAR	22,896,000	100	44	44%
LCH-4007	150	50	145	51	8	0.066	REGULAR	23,928,000	150	46	31%
LCH-4009	134	UNK	UNK	50	8	0.207	REGULAR	75,390,000	350	144	41%
LCH-5186	160	UNK	UNK	50	10	0.276	REGULAR	100,653,000	350	192	55%
614 (planned)	188	118	178	100	10	(planned well)	REGULAR		250	0	0%
621 (planned)	180	120	170	100	10	(planned well)	REGULAR		175	0	0%
622 (planned)	175	105	165	90	10	(planned well)	REGULAR		380	0	0%
							TOTAL	1,166,499,360			

(existing wells) 5,878 GPM TOTAL PUMPING CAPACITY
(existing wells) 8,464,320 GPD TOTAL PUMPING CAPACITY

(existing + expansion wells) 6,683 GPM TOTAL PUMPING CAPACITY
(existing + expansion wells) 9,623,520 GPD TOTAL PUMPING CAPACITY

Average daily withdrawal from planned wells will be estimated at pumping capacity over 14 hours/day: 676,200 GPD estimated average
Maximum daily withdrawal from planned wells will be estimated at pumping capacity over 18 hours/day: 869,400 GPD estimated max

TABLE 4-1 (cont'd)
Pumping Rates of Water Supply Wells at MCB, Camp Lejeune

Well Name* or Number	Well Depth (feet)	Screen Depth Top (feet)	Screen Depth Bottom (feet)	Casing Depth (feet)	Well Diameter (inches)	Average Daily Withdrawal in 1993 (MGD)	Regular or Emergency	TOTAL GALLONS* (1993)	MAX PUMPING RATE (GPM)	AVG PUMPING RATE (GPM)	% TIME ON
643	240	90	240	88	10	0.235	REGULAR	85,671,120	269	163	61%
644	255	85	250	85	10	0.217	REGULAR	79,253,400	230	151	66%
646	266	90	265	50	10	0.219	REGULAR	79,764,000	425	152	36%
647	200	105	190	105	10	0.33	REGULAR	120,316,800	105	229	218%
648	260	107	260	107	10	0.016	REGULAR	5,989,200	280	11	4%
650	179	128	174	50	10	0.044	REGULAR	16,168,320	480	31	6%
698	124	84	124	50	10	0.228	REGULAR	83,374,800	244	158	65%
699	108	72	98	50	10	0.026	REGULAR	9,595,980	267	18	7%
700	130	100	130	50	10	0.114	REGULAR	41,756,400	140	79	57%
701	100	70	100	50	10	0.147	REGULAR	53,540,160	172	102	59%
703	145	75	145	50	10	0.18	REGULAR	65,756,160	192	125	65%
704	124	84	114	50	10	0.122	REGULAR	44,475,480	159	85	53%
705	160	120	160	50	10	0.172	REGULAR	62,904,420	185	119	65%
706	185	126	176	50	10	0.183	REGULAR	66,935,640	199	127	64%
707	130	80	130	50	10	0.062	REGULAR	22,581,000	130	43	33%
708	176	126	176	50	10	0.222	REGULAR	81,100,080	219	154	70%
617 (planned)	265	205	255	UNK	10	(planned well)	REGULAR		410	0	0%
618 (planned)	240	180	230	UNK	6	(planned well)	REGULAR		600	0	0%
619 (planned)	211	125	201	100	10	(planned well)	REGULAR		175	0	0%
							TOTAL	919,182,960			

	for existing wells:
3,696	GPM TOTAL PUMPING CAPACITY
5,322,240	GPD TOTAL PUMPING CAPACITY
	for existing + expansion:
4,881	GPM TOTAL PUMPING CAPACITY
7,028,640	GPD TOTAL PUMPING CAPACITY

* estimated by operators from logs

Average daily withdrawal from planned wells will be estimated at pumping capacity over 14 hours	995,400	GPD estimated average
Maximum daily withdrawal from planned wells will be estimated at pumping capacity over 18 hour	1,279,800	GPD estimated max

TABLE 4-1 (cont'd)
Pumping Rates of Water Supply Wells at MCB, Camp Lejeune

Well Name* or Number	Well Depth (feet)	Screen Depth Top (feet)	Screen Depth Bottom (feet)	Casing Depth (feet)	Well Diameter (inches)	Average Daily Withdrawal in 1993 (MGD)	Regular or Emergency	TOTAL GALLONS* (1993)	MAX PUMPIN RATE (GPM)	AVG PUMPING RATE (GPM)	% TIME ON
AS-106	179	UNK	UNK	UNK	8	0.006	REGULAR	2,020,320	183	4	2%
AS-131	200	UNK	UNK	UNK	8	0.002	REGULAR	576,600	310	1	0%
AS-190	180	UNK	UNK	UNK	8	0.095	REGULAR	34,620,900	190	66	35%
AS-191	180	UNK	UNK	60	8	0.082	REGULAR	29,779,200	281	57	20%
AS-203	173	UNK	UNK	UNK	8	0.010	REGULAR	3,604,400	220	7	3%
AS-4140	193	UNK	UNK	UNK	8	0.0000	REGULAR	178,200	110	0	0%
AS-4150	193	UNK	UNK	UNK	8	0.001	REGULAR	245,760	128	1	1%
AS-5001	193	UNK	UNK	UNK	8	0.171	REGULAR	62,382,000	130	119	91%
AS-5009	196	UNK	UNK	UNK	8	0.047	REGULAR	16,996,320	111	33	29%
TC-502	184	110	184	110	10	0.001	REGULAR	211,500	400	1	0%
TC-600	70	48	70	70	8	0.148	REGULAR	54,056,160	104	103	99%
TC-604	113	45	113	45	8	0.161	REGULAR	58,607,040	154	112	73%
TC-700	76	27.5	76	27.5	18	0.029	REGULAR	10,725,000	125	20	16%
TC-1000	153	86	136	86	8	0.075	REGULAR	27,475,800	104	52	50%
TC-1001	100	70	100	70	8	0.131	REGULAR	47,942,400	160	91	57%
TC-1251	155	120	140	UNK	UNK	0.002	REGULAR	621,000	175	1	1%
TC-1253	250	120	170	50	UNK	0.102	REGULAR	37,309,440	128	71	55%
TC-1254	195	118	160	UNK	UNK	0.082	REGULAR	29,760,000	100	57	57%
TC-1255	250	124	190	UNK	UNK	0.040	REGULAR	14,720,160	50	28	56%
TC-1256	204	124	192	60	UNK	0.060	REGULAR	21,840,000	108	42	39%
							TOTAL	453,672,200	3271	GPM TOTAL PUMPING CAPACITY	
									4710240	GPD TOTAL PUMPING CAPACITY	

* estimated by operators from logs

Average daily withdrawal from planned wells will be estimated at pumping capacity over 14 hours/day:
Maximum daily withdrawal from planned wells will be estimated at pumping capacity over 18 hours/day:

TABLE 4-2 -- STATISTICAL SUMMARY OF BRAGS SIMULATION
BRAGS, CTO-0140
MCB, CAMP LEJUENE, NORTH CAROLINA

Location Name	Target Value	Computed Value	Error	Error Summary	
			Layer 1		
BB-220	27.00	21.63	-5.37	Mean Error	-1.08
Site 1	8.56	9.26	0.70	Mean Absolute Error	3.48
Site 2	28.50	36.02	7.52	Root Mean Square Error	5.37
Site 3	24.00	31.89	7.89	Standard Deviation of Errors	5.38
Site 6	16.74	14.88	-1.86	Maximum Error	7.89
Site 7	4.30	5.55	1.25	Minimum Error	-18.69
Site 9	17.75	11.33	-6.42		
Site 16	3.50	4.12	0.62		
Site 21	20.00	16.80	-3.20		
Site 24	8.50	14.56	6.06		
Site 28	2.33	2.35	0.02		
Site 30	34.26	28.33	-5.93		
Site 35	8.80	5.50	-3.30		
Site 36	3.84	4.14	0.30		
Site 41	9.86	5.67	-4.19		
Site 43	1.35	1.89	0.54		
Site 44	4.51	6.27	1.76		
Site 69	25.39	6.70	-18.69		
Site 73	8.54	6.96	-1.58		
Site 74	14.50	14.00	-0.50		
Site 78	15.33	15.70	0.37		
Site 86	9.86	8.42	-1.44		
UST 21	3.30	3.83	0.53		
			Layer 3		
A-5	4.41	2.70	-1.71	Mean Error	-1.68
BA-164	11.00	10.57	-0.43	Mean Absolute Error	2.57
BA-190	7.00	6.44	-0.56	Root Mean Square Error	4.89
BB-43	1.20	1.57	0.37	Standard Deviation of Errors	4.68
BB-44	3.70	2.45	-1.25	Maximum Error	6.27
BB-45	3.00	1.65	-1.35	Minimum Error	-22.58
CCC-1	3.00	5.06	2.06		
CCC-2	7.00	8.28	1.28		
LCH-4006	12.00	7.35	-4.65		
LCH-4007	14.00	11.92	-2.08		
RR-45	4.00	10.27	6.27		
TC-201	17.00	14.15	-2.85		
TC-202	16.00	13.92	-2.08		
TC-604	22.00	14.66	-7.34		

TABLE 4-2 -- STATISTICAL SUMMARY OF BRAGS SIMULATION
BRAGS, CTO-0140
MCB, CAMP LEJUENE, NORTH CAROLINA

Location Name	Target Value	Computed Value	Error	Error Summary	
			Layer 3 (continued)		
Site 1D	7.25	8.52	1.27		
Site 2D	1.50	1.45	-0.05		
Site 3D	7.00	7.11	0.11		
Site 6D	12.39	9.07	-3.32		
Site 9D	10.00	8.91	-1.09		
Site 28D	2.33	3.04	0.71		
Site 35D	8.67	7.22	-1.45		
Site 36D	5.00	4.10	-0.90		
Site 41D	9.88	5.77	-4.11		
Site 43D	2.22	2.33	0.11		
Site 44D	5.98	6.57	0.59		
Site 69D	28.03	5.45	-22.58		
Site 73D	4.00	3.43	-0.57		
Site 78D	15.25	13.81	-1.44		
Site 86D	10.00	8.20	-1.80		
			Layer 5		
HP-601	-0.14	10.57	10.71	Mean Error	-0.52
HP-602	15.00	12.53	-2.47	Mean Absolute Error	5.33
HP-606	16.70	17.02	0.32	Root Mean Square Error	6.66
HP-609	23.00	16.08	-6.92	Standard Deviation of Errors	6.68
HP-610	5.00	10.81	5.81	Maximum Error	12.93
HP-612	9.60	3.37	-6.23	Minimum Error	-19.24
HP-613	10.00	5.66	-4.34		
HP-614	13.20	0.45	-12.75		
HP-615	16.00	0.15	-15.85		
HP-626	13.30	14.51	1.21		
HP-628	14.00	15.87	1.87		
HP-629	20.00	20.70	0.70		
HP-630	12.00	14.32	2.32		
HP-633	15.00	6.41	-8.59		
HP-634	20.00	16.35	-3.65		
HP-635	6.00	13.76	7.76		
HP-636	16.00	13.45	-2.55		
HP-637	13.00	13.11	0.11		
HP-638	4.00	8.65	4.65		
HP-639	19.00	17.24	-1.76		
HP-640	27.00	23.12	-3.88		
HP-641	12.00	13.30	1.30		

TABLE 4-2 -- STATISTICAL SUMMARY OF BRAGS SIMULATION
BRAGS, CTO-0140
MCB, CAMP LEJUENE, NORTH CAROLINA

Location Name	Target Value	Computed Value	Error	Error Summary
			Layer 5 (continued)	
HP-642	19.00	17.64	-1.36	
HP-643	12.00	-0.05	-12.05	
HP-644	5.00	-0.53	-5.53	
HP-645	9.00	0.81	-8.19	
HP-646	3.00	2.01	-0.99	
HP-647	15.00	4.24	-10.76	
HP-648	26.00	25.83	-0.17	
HP-649	21.00	29.37	8.37	
HP-650	26.00	31.60	5.60	
HP-651	25.00	10.89	-14.11	
HP-652	23.00	22.81	-0.19	
HP-653	17.00	11.21	-5.79	
HP-654	17.00	9.58	-7.42	
HP-655	15.00	12.79	-2.21	
HP-661	12.00	13.14	1.14	
HP-663	15.00	27.93	12.93	
HP-698	13.00	0.06	-12.94	
HP-699	12.00	-0.27	-12.27	
HP-700	4.00	0.95	-3.05	
HP-701	6.00	3.59	-2.41	
HP-703	21.00	1.76	-19.24	
HP-704	5.00	3.43	-1.57	
HP-705	12.00	9.91	-2.09	
HP-706	22.00	15.87	-6.13	
HP-708	31.00	33.06	2.06	
HP-709	13.00	15.14	2.14	
HP-710	12.00	16.64	4.64	
HP-711	27.00	21.65	-5.35	
M-161	2.00	10.66	8.66	
M-168	8.00	10.69	2.69	
M-197	9.00	8.23	-0.77	
M-267	3.00	5.21	2.21	
M-628	6.00	9.82	3.82	
M-629	5.00	6.62	1.62	
M-630	5.00	7.27	2.27	
MCAS-106	1.00	8.84	7.84	
MCAS-131	1.00	8.76	7.76	
MCAS-203	3.00	8.51	5.51	
MCAS-4140	2.00	10.31	8.31	
NC-52	3.00	10.55	7.55	

TABLE 4-2 – STATISTICAL SUMMARY OF BRAGS SIMULATION
BRAGS, CTO-0140
MCB, CAMP LEJUENE, NORTH CAROLINA

Location Name	Target Value	Computed Value	Error	Error Summary
		Layer 5 (continued)		
OW-2	6.00	6.13	0.13	
OW-3	14.00	5.80	-8.20	
OW-4	7.00	13.54	6.54	
OW-5	8.00	7.99	-0.01	
RR-47	0.00	7.55	7.55	
RR-97	20.00	12.02	-7.98	
RR-229	19.00	8.48	-10.52	
T-9	21.00	16.04	-4.96	
TC-100	14.50	11.12	-3.38	
TC-104	4.00	11.32	7.32	
TC-191	1.00	9.44	8.44	
TC-600	13.00	8.56	-4.44	
TC-901	15.80	9.01	-6.79	
TC-1001	23.20	10.99	-12.21	
TC-1253	5.00	8.85	3.85	
TC-1255	10.00	9.16	-0.84	
TC-1256	8.00	10.94	2.94	
TT-23/25	6.00	10.78	4.78	
TT-26	7.00	12.83	5.83	
TT-31	2.00	8.99	6.99	
TT-52	3.00	10.00	7.00	
TT-53	8.00	12.53	4.53	
TT-54	3.00	8.34	5.34	
TT-67	6.00	10.78	4.78	
X (1950)	14.00	15.23	1.23	
X24c2	8.00	12.31	4.31	
X24s2x	5.00	4.76	-0.24	
Y25q2	34.00	36.80	2.80	

Layers 1, 3, & 5

Mean Error	-0.85
Mean Absolute Error	4.46
Root Mean Square Error	6.14
Standard Deviation of Errors	6.10
Maximum Error	12.93
Minimum Error	-22.58

TABLE 4-3 -- HYDROLOGIC BUDGET SUMMARY FOR BRAGS SIMULATION
BRAGS, CTO-0140
MCB, CAMP LEJEUNE, NORTH CAROLINA

LAYER	Discharge to Atlantic Ocean (gpm)	Discharge to Supply Wells (gpm)	Discharge to Streams & Creeks (gpm)	Discharge to New River (gpm)	Flow through Lateral Boundaries (gpm)	Recharge 11 in/yr (gpm)		
1	-1,542	0	-45,125	-36,171	1,417	95,931	<==>	18,468,000 cfd
2	0	0	0	0	0	0		
3	0	0	0	0	-3,056	0		
4	0	-5,366	0	0	-3,042	0		
5	0	0	0	0	-3,042	0		

TOTAL IN:	97,348
TOTAL OUT	-97,343
Difference	5
% Error	0.00

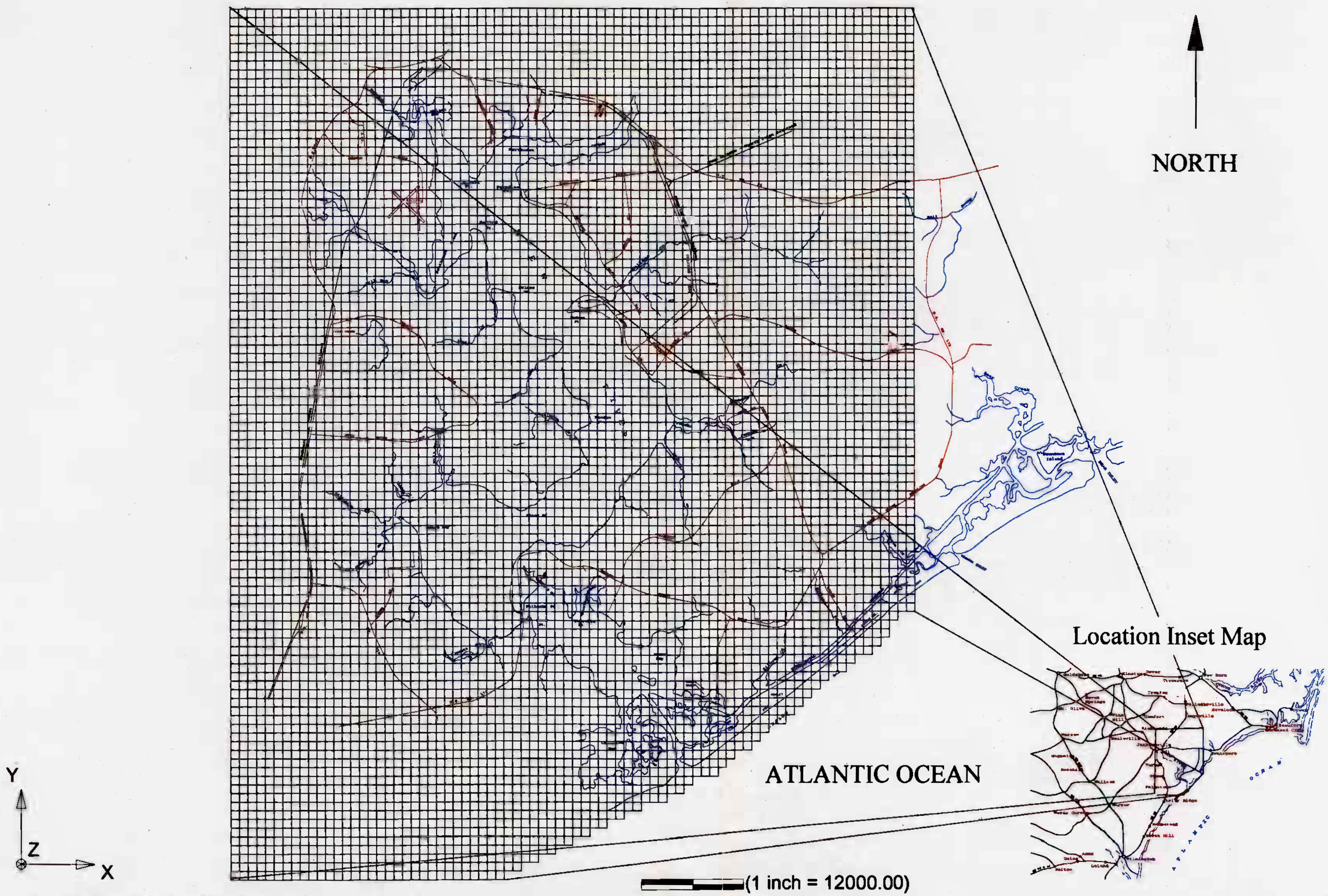
INFILTRATION INTO THE CASTLE HAYNE:

Net flow from Layer 1 into Layer 3	14,510	gpm
which is lost to lateral boundaries in Layers 3, 4, & 5	2,793,404	cfd
(sum of all Layer 1 rows) =========>	1.66	in / yr

All values taken from the MODFLOW output file (brags-cbc.out -- see Appendix B)
Negative values indicate water discharging from (exiting) the model.
Positive values indicate water recharging (entering) the model.

SECTION 4.0 FIGURES

FIGURE 4-1 Finite-Difference Grid Location Map
CTO-0140 BRAGS Groundwater Flow Model
MCB, Camp Lejeune, North Carolina



02013AA01Y

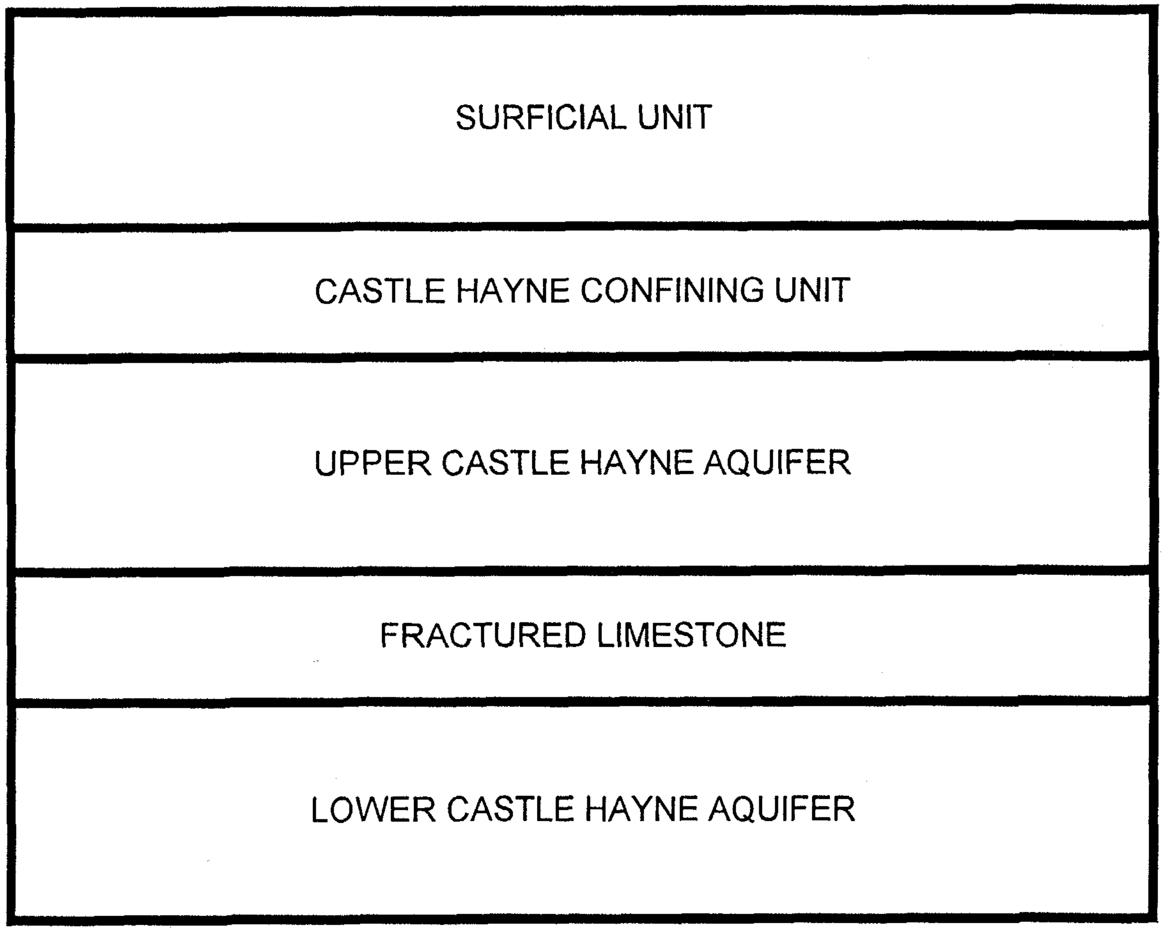


FIGURE 4-2 Schematic of 3-D Five-Layer BRAGS Model

FIGURE 4-3 Computed vs. Observed Values in Layer 1
BRAGS Model -- MCB, Camp Lejuene, North Carolina

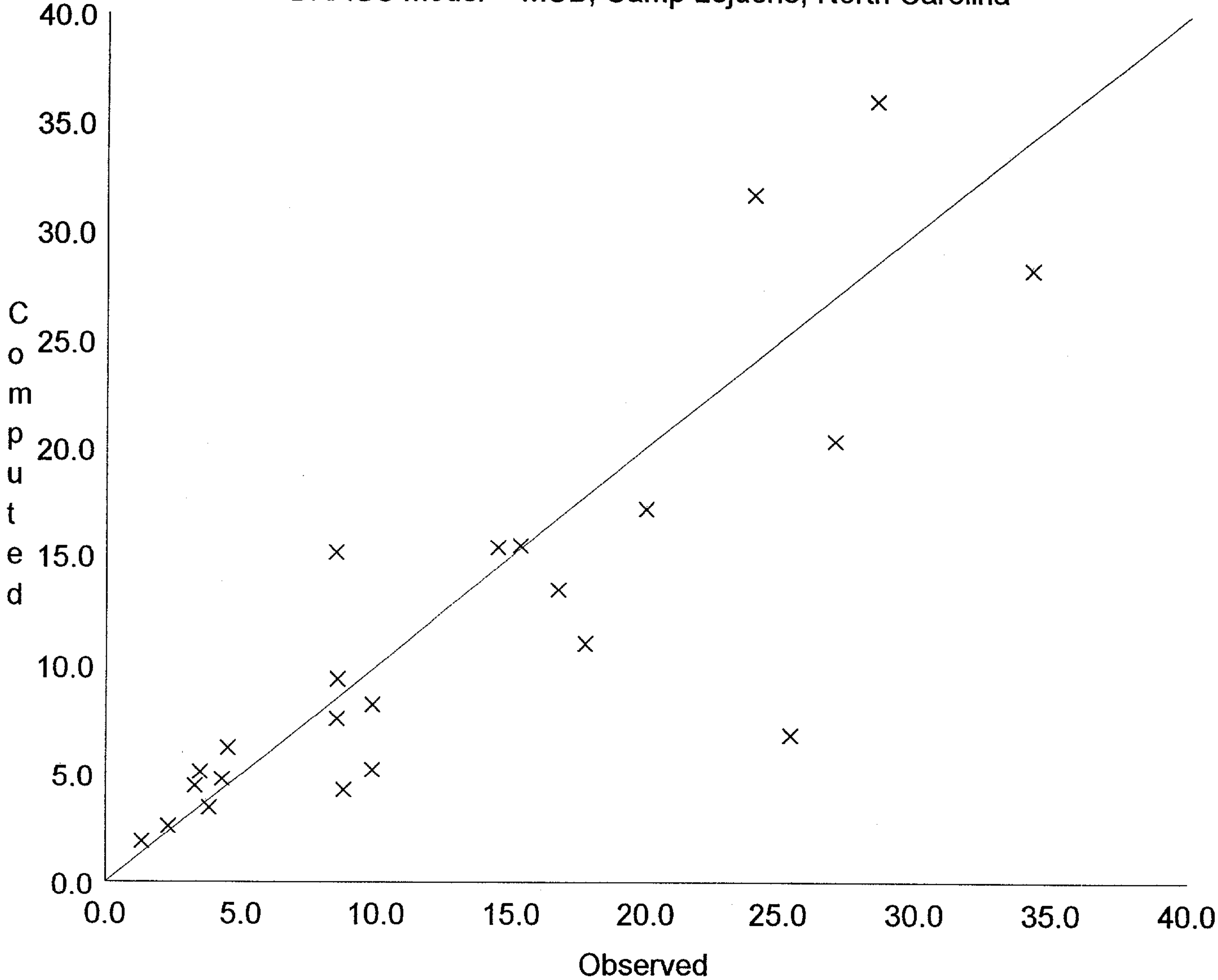


FIGURE 4-4 Error vs. Observed Values in Layer 1

BRAGS, MCB, Camp Lejeune, North Carolina

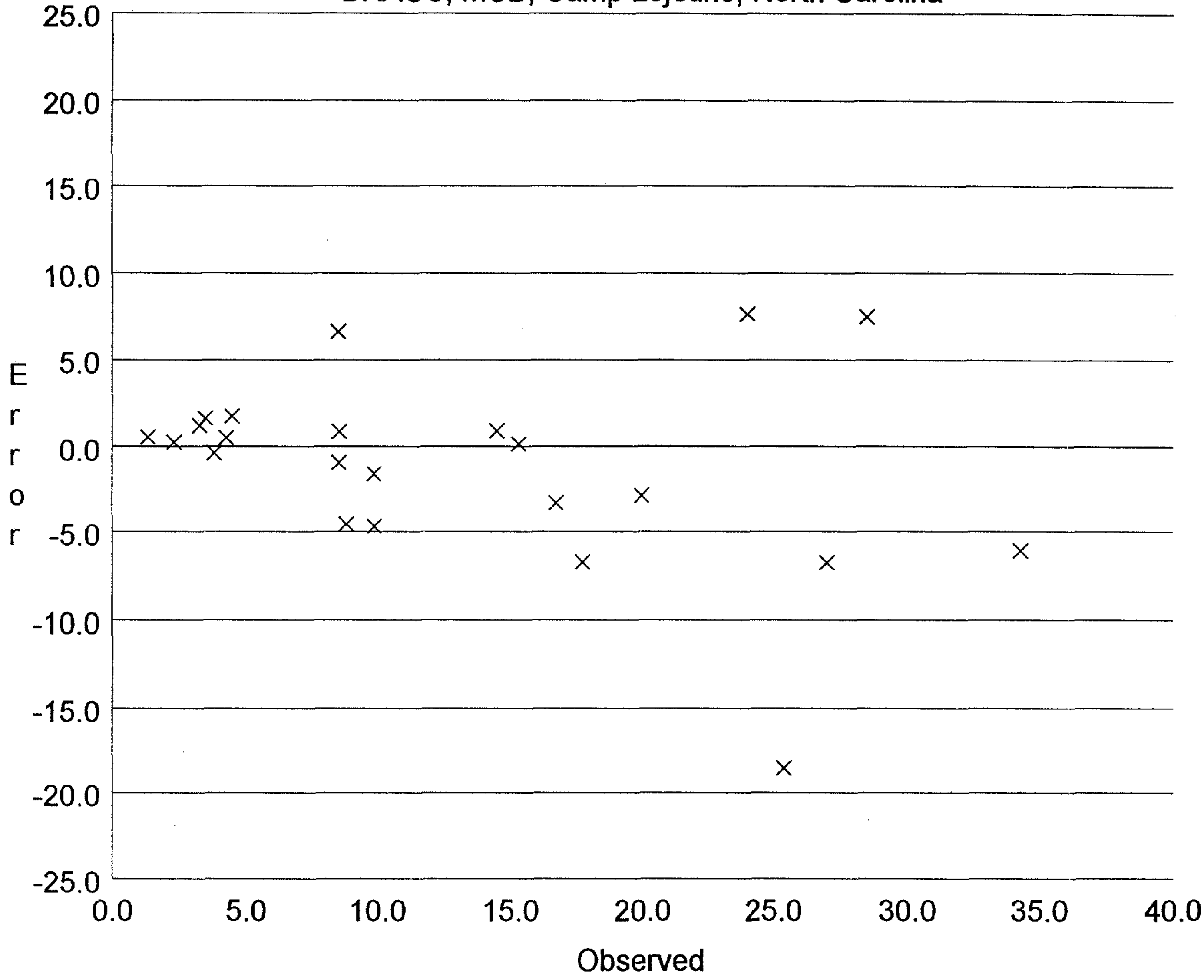


FIGURE 4-5 Computed vs. Observed Values in Layer 3
BRAGS Model -- MCB, Camp Lejeune, North Carolina

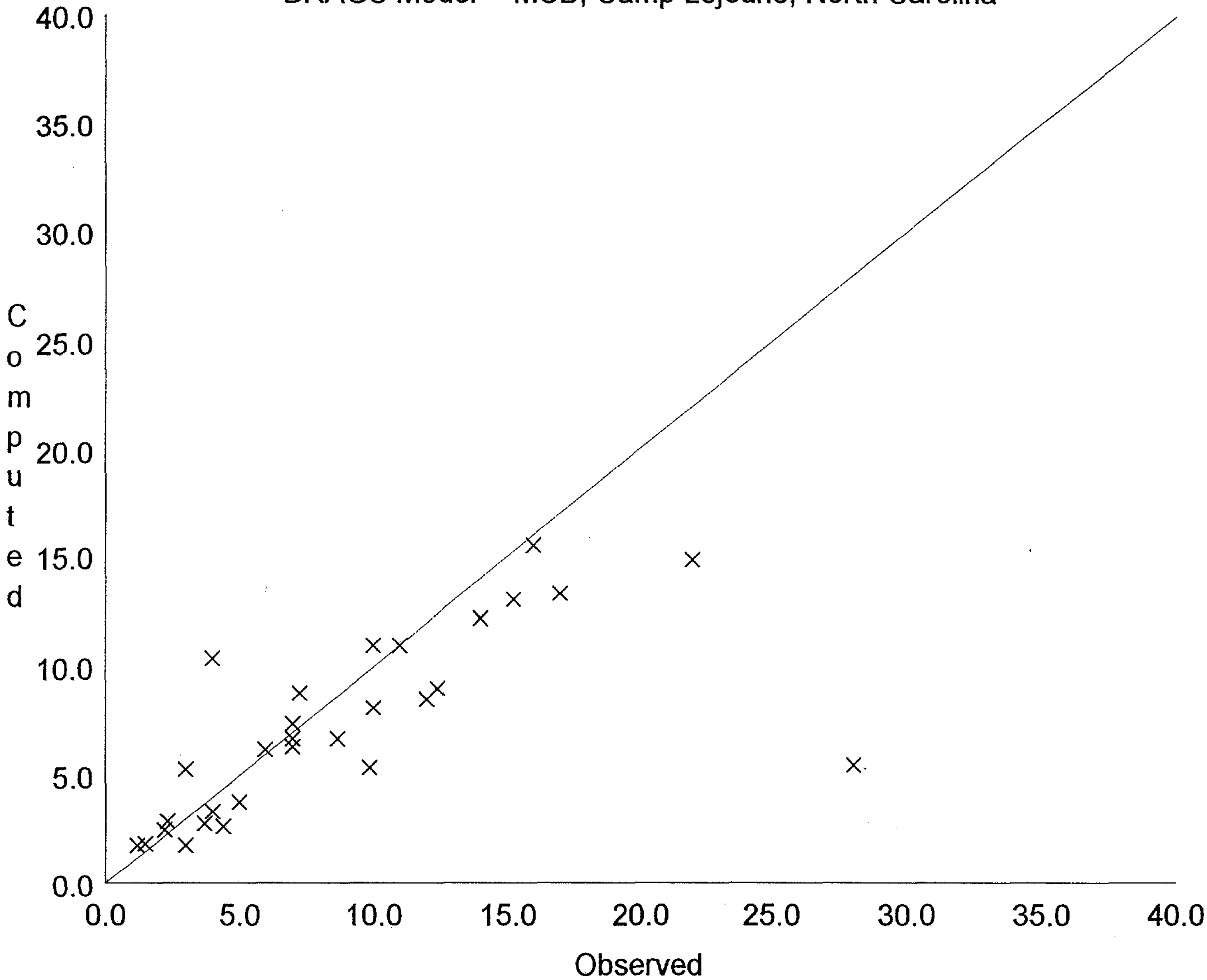


FIGURE 4-6 Error vs. Observed Values in Layer 3

BRAGS, MCB, Camp Lejeune, North Carolina

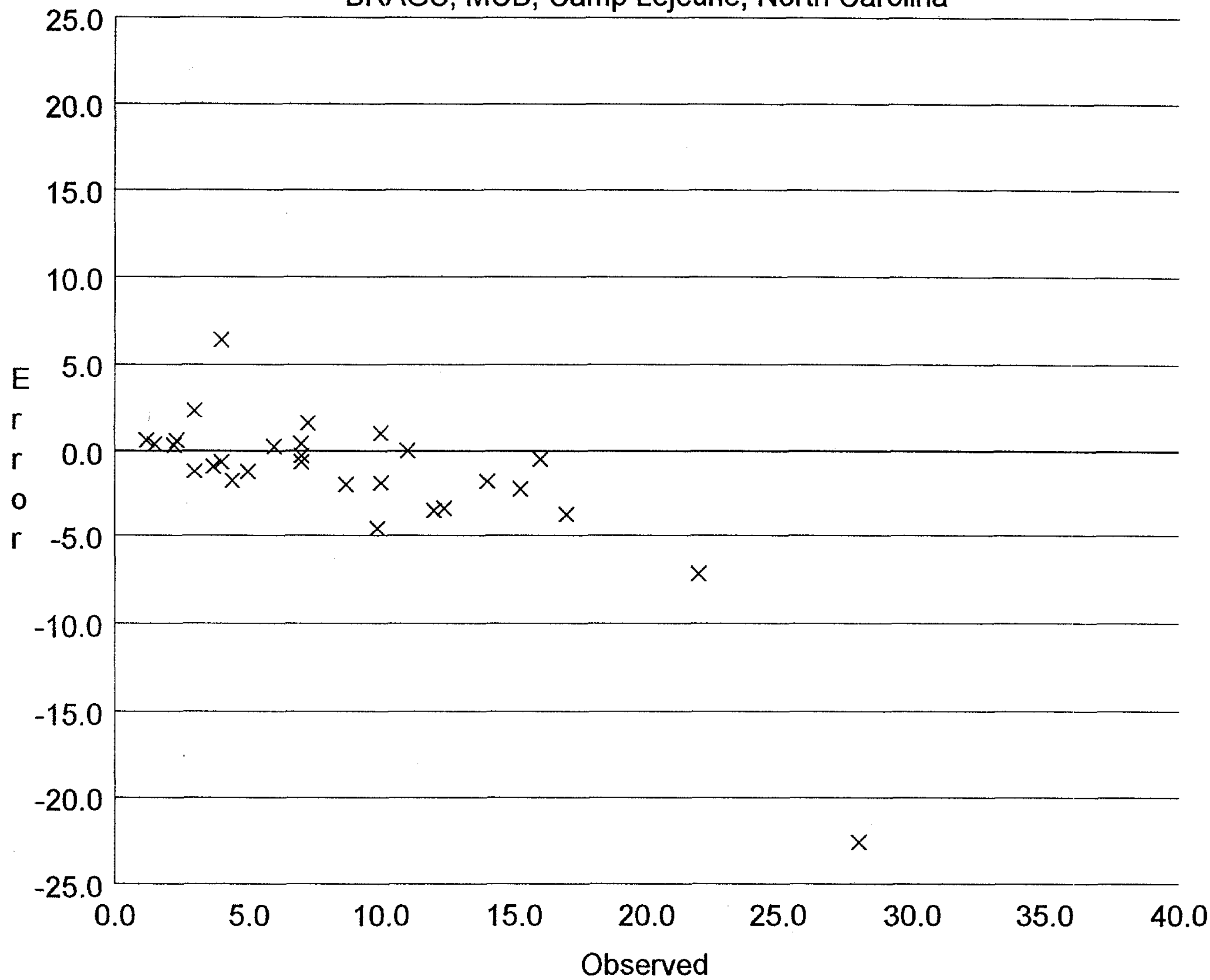


FIGURE 4-7 Computed vs. Observed Values in Layer 5
BRAGS Model -- MCB, Camp Lejeune, North Carolina

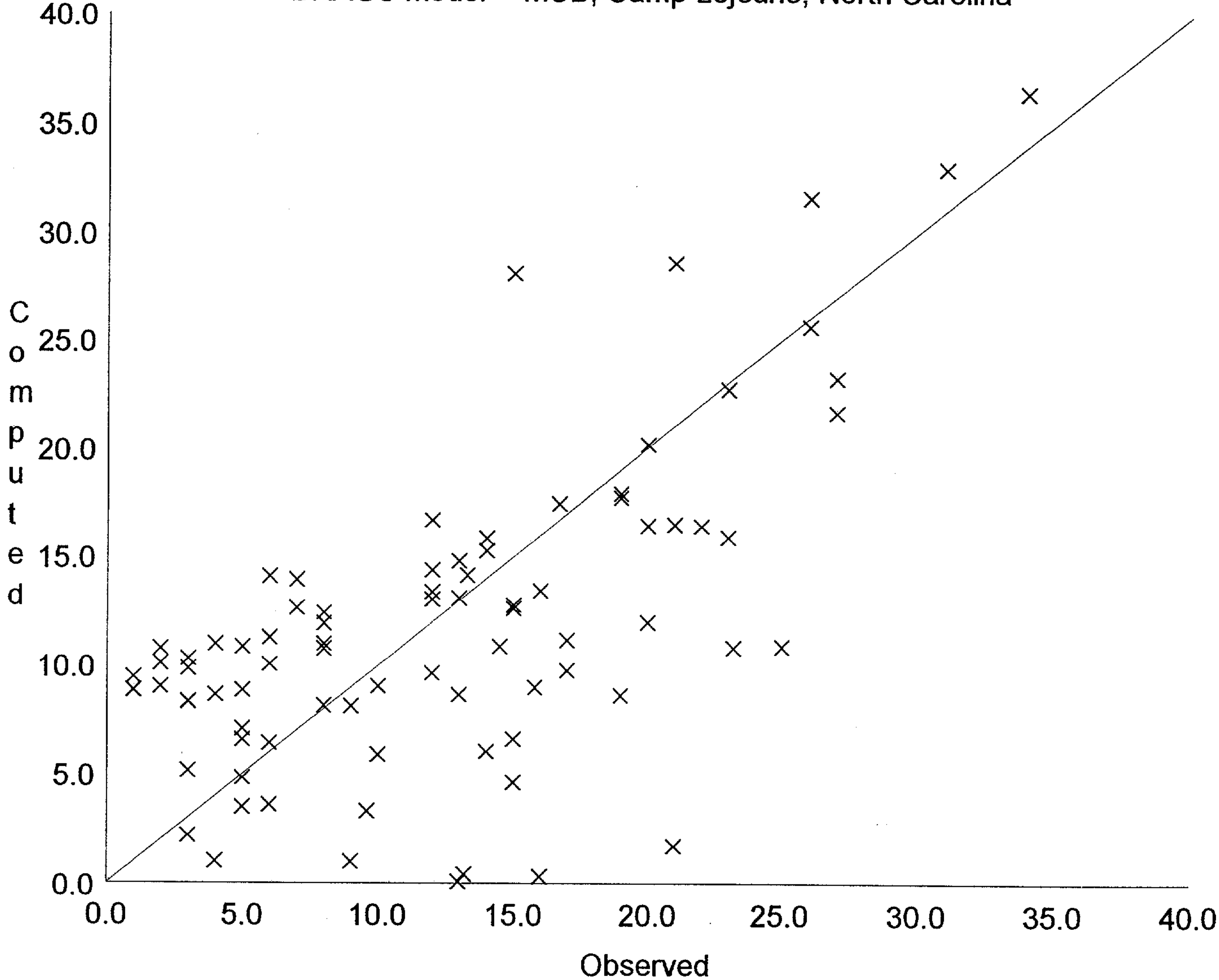
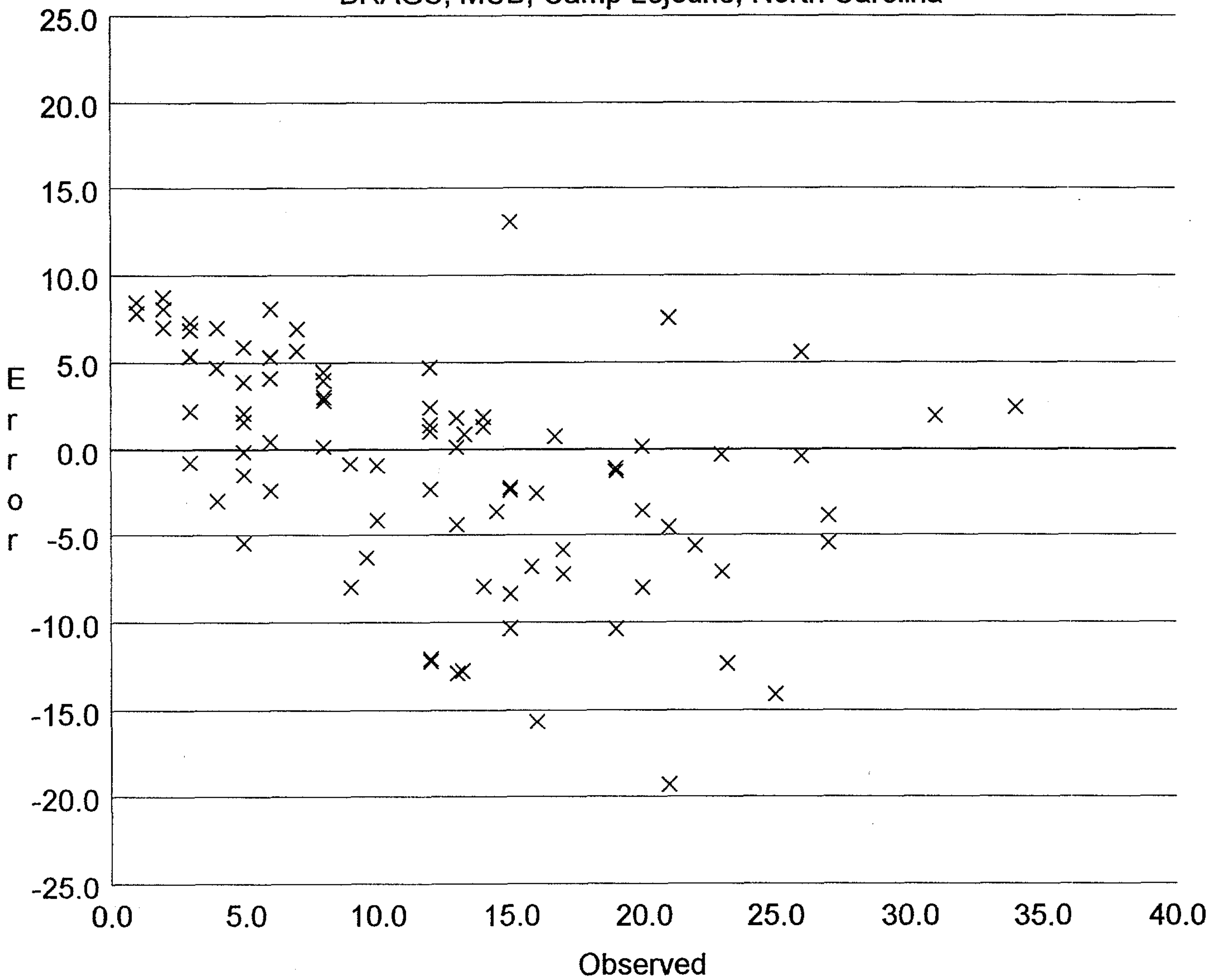
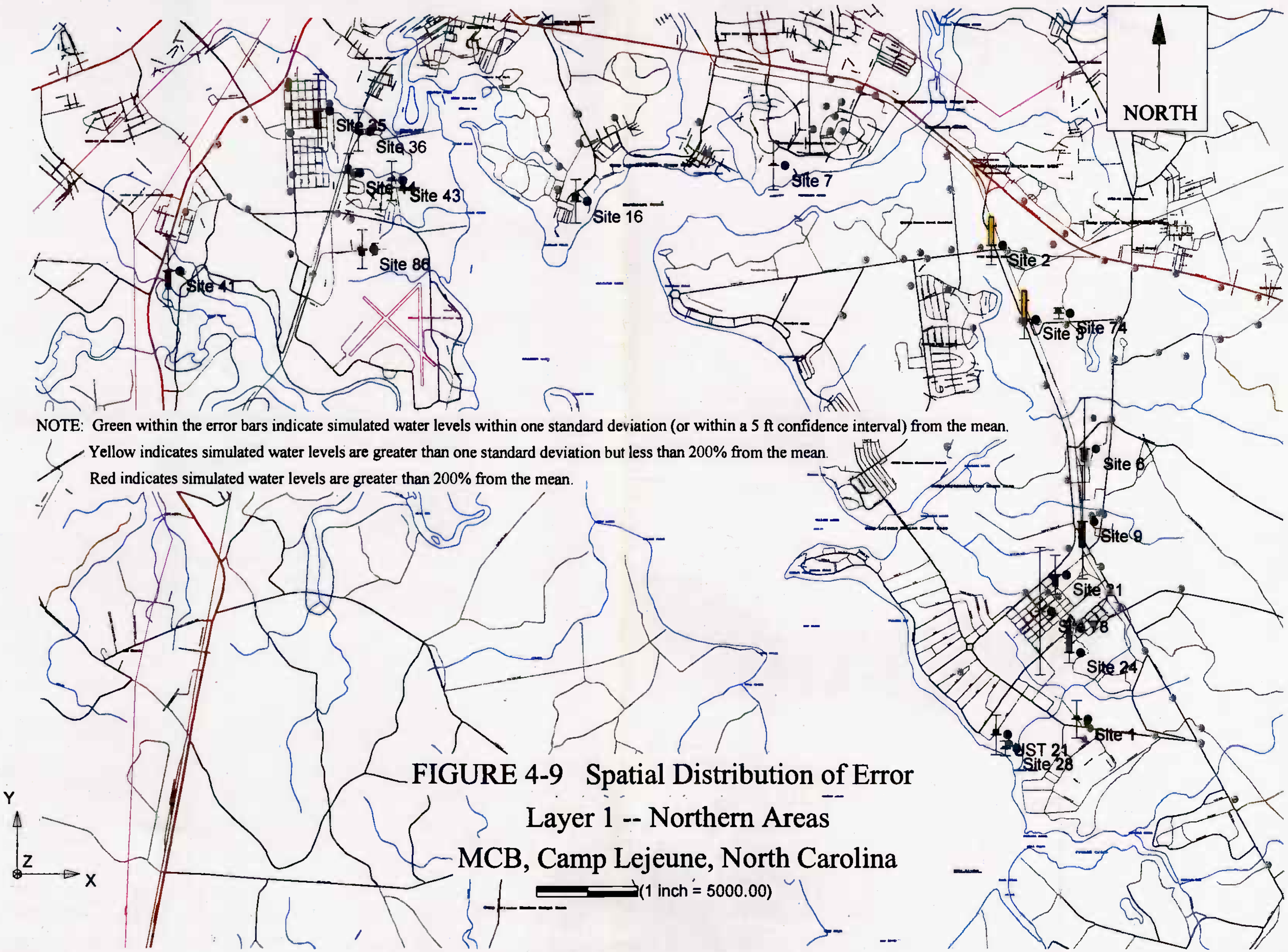


FIGURE 4-8 Error vs. Observed Values in Layer 5
BRAGS, MCB, Camp Lejeune, North Carolina





NOTE: Green within the error bars indicate simulated water levels within one standard deviation (or within a 5 ft confidence interval) from the mean.
Yellow indicates simulated water levels are greater than one standard deviation but less than 200% from the mean.
Red indicates simulated water levels are greater than 200% from the mean.

FIGURE 4-9 Spatial Distribution of Error
Layer 1 -- Northern Areas
MCB, Camp Lejeune, North Carolina

02013AAB2X

NOTE: Green within the error bars indicate simulated water levels within one standard deviation (or within a 5 ft confidence interval) from the mean.
Yellow indicates simulated water levels are greater than one standard deviation but less than 200% from the mean.
Red indicates simulated water levels are greater than 200% from the mean.

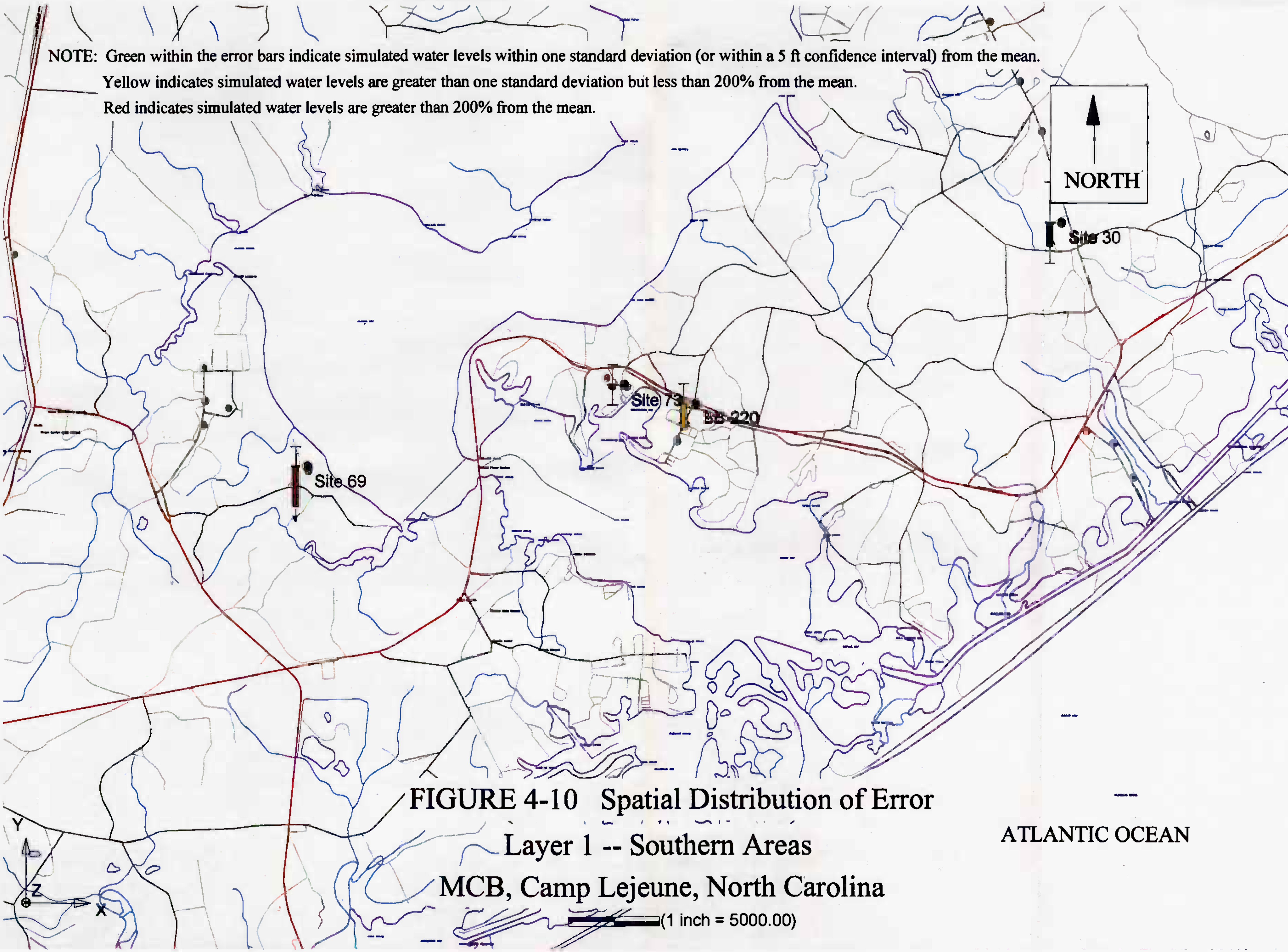
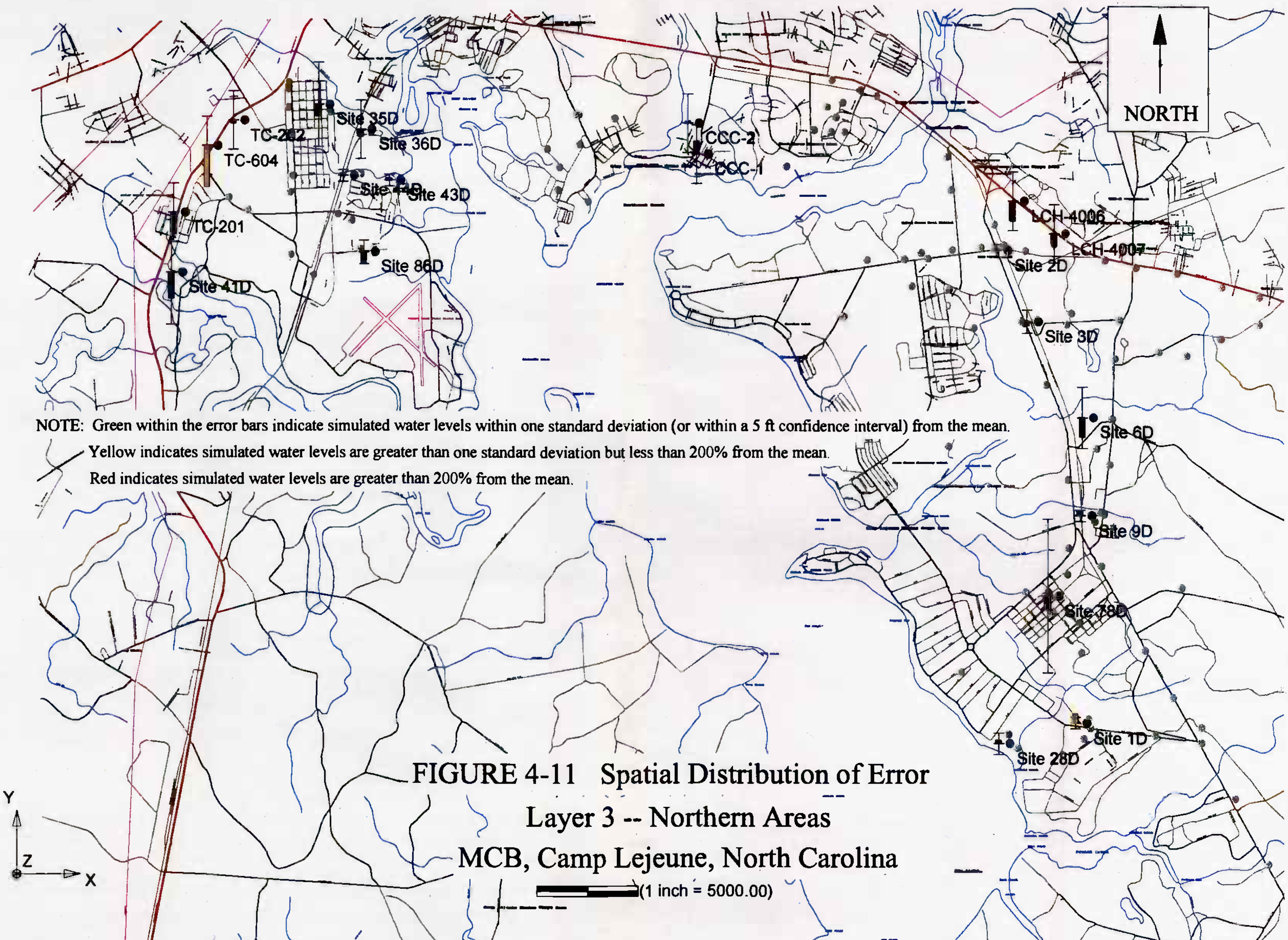


FIGURE 4-10 Spatial Distribution of Error
Layer 1 -- Southern Areas
MCB, Camp Lejeune, North Carolina



NOTE: Green within the error bars indicate simulated water levels within one standard deviation (or within a 5 ft confidence interval) from the mean.
Yellow indicates simulated water levels are greater than one standard deviation but less than 200% from the mean.
Red indicates simulated water levels are greater than 200% from the mean.

FIGURE 4-11 Spatial Distribution of Error
Layer 3 -- Northern Areas
MCB, Camp Lejeune, North Carolina

NOTE: Green within the error bars indicate simulated water levels within one standard deviation (or within a 5 ft confidence interval) from the mean.
Yellow indicates simulated water levels are greater than one standard deviation but less than 200% from the mean.
Red indicates simulated water levels are greater than 200% from the mean.

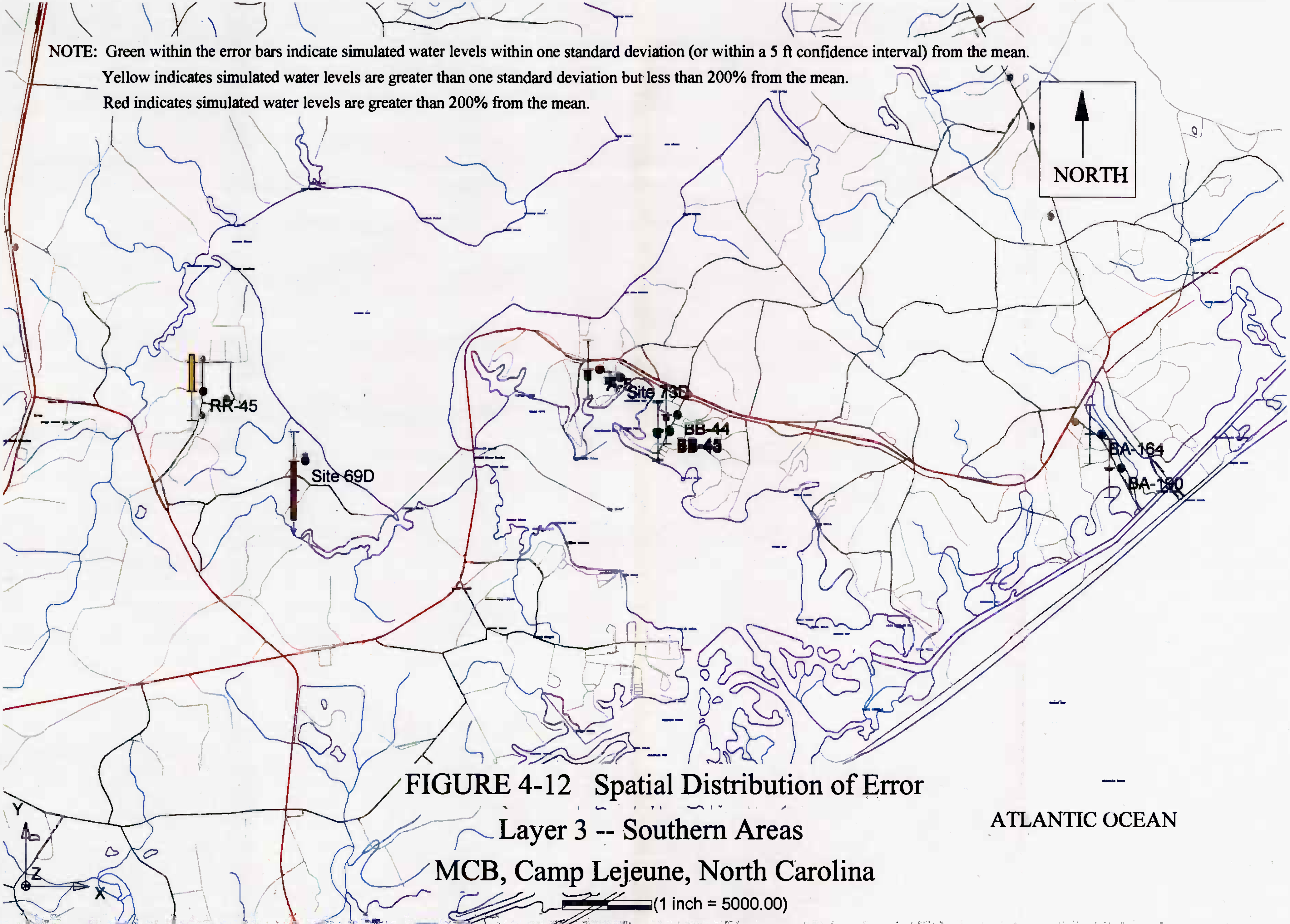
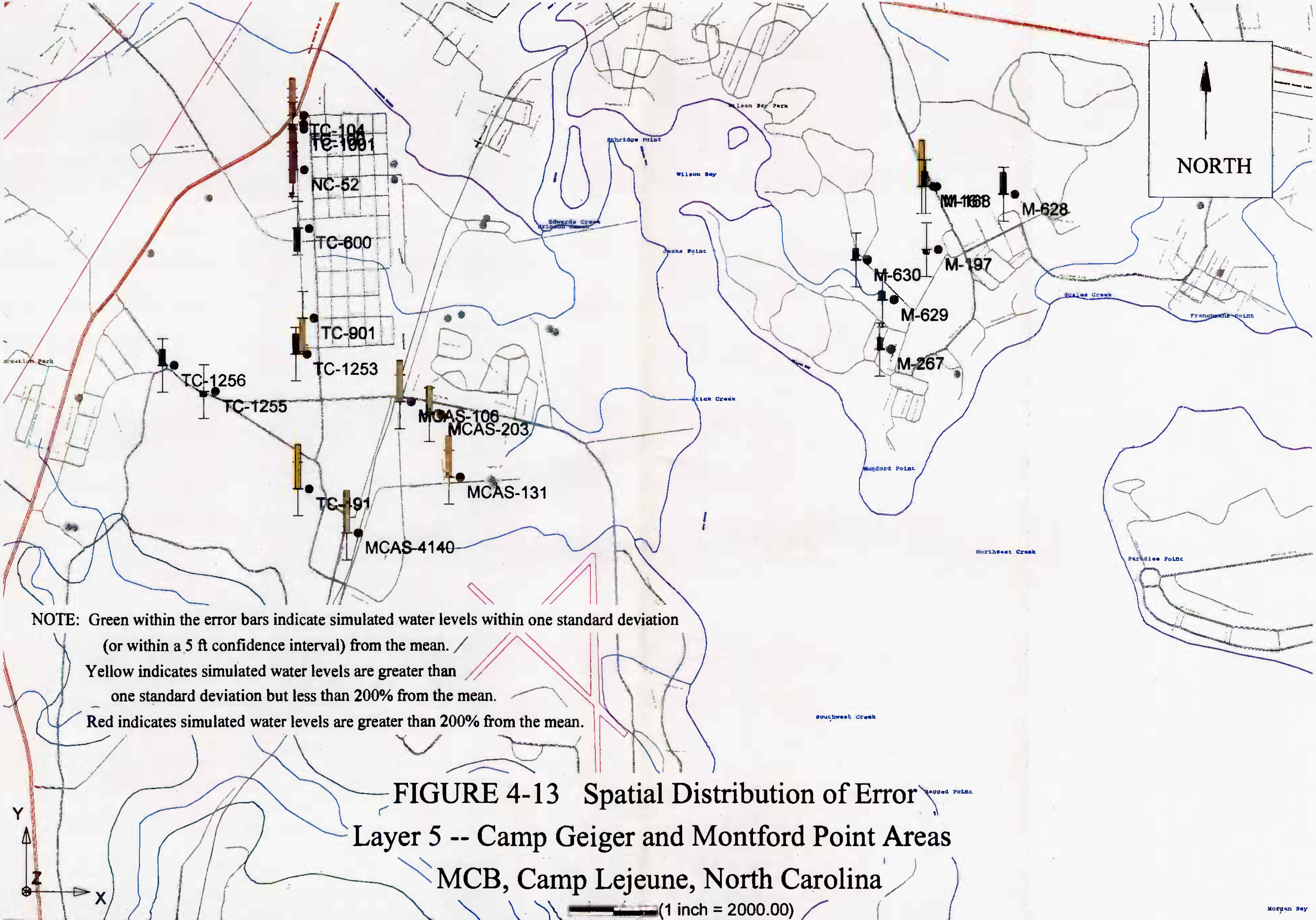


FIGURE 4-12 Spatial Distribution of Error
Layer 3 -- Southern Areas
MCB, Camp Lejeune, North Carolina



NOTE: Green within the error bars indicate simulated water levels within one standard deviation (or within a 5 ft confidence interval) from the mean.

Yellow indicates simulated water levels are greater than one standard deviation but less than 200% from the mean.

Red indicates simulated water levels are greater than 200% from the mean.

FIGURE 4-13 Spatial Distribution of Error

Layer 5 -- Camp Geiger and Montford Point Areas

MCB, Camp Lejeune, North Carolina

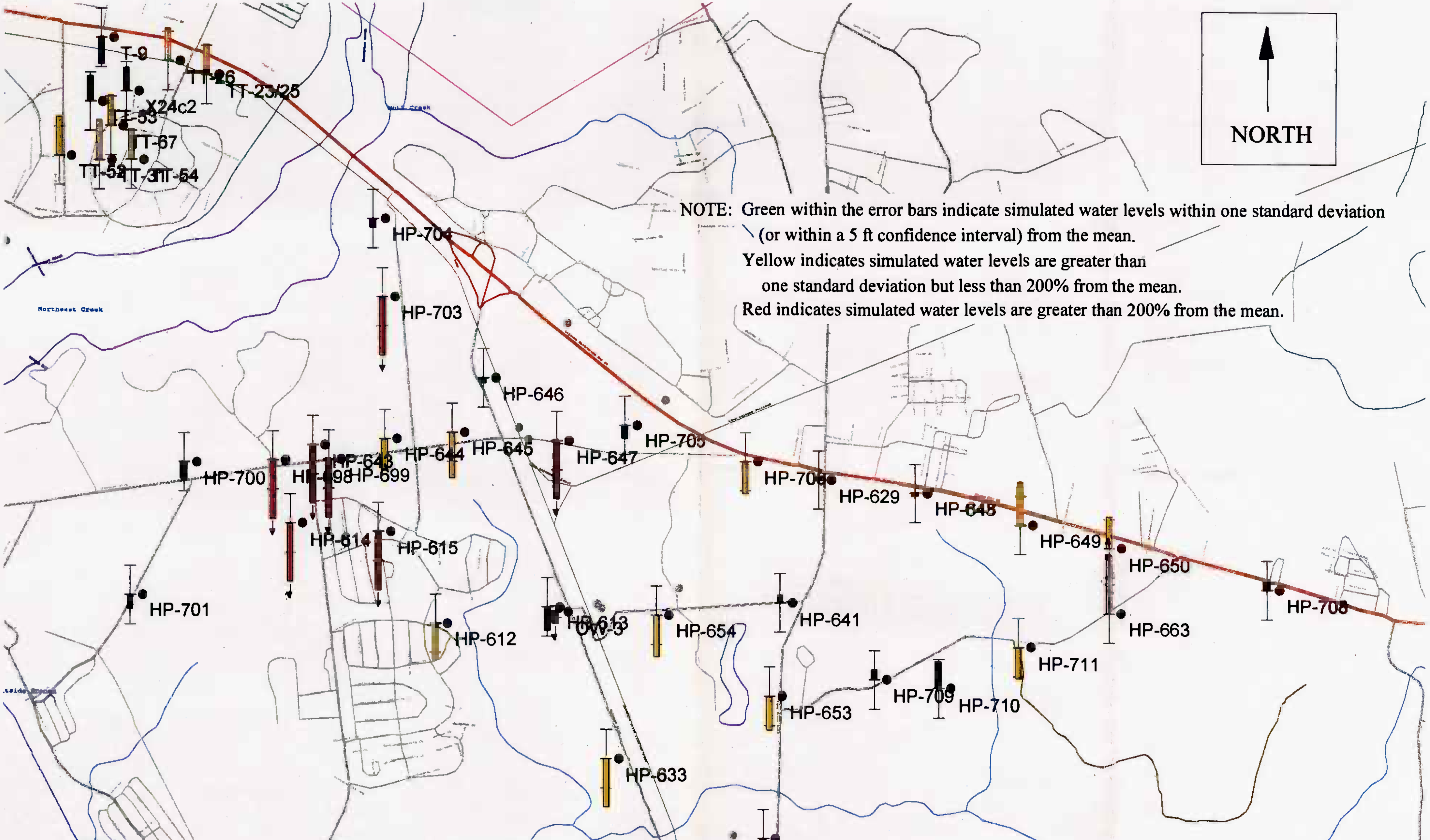


FIGURE 4-14 Spatial Distribution of Error

Layer 5 -- Tarawa Terrace and Paradise Point Areas

MCB, Camp Lejeune, North Carolina

NOTE: Green within the error bars indicate simulated water levels within one standard deviation (or within a 5 ft confidence interval) from the mean.
Yellow indicates simulated water levels are greater than one standard deviation but less than 200% from the mean.
Red indicates simulated water levels are greater than 200% from the mean.

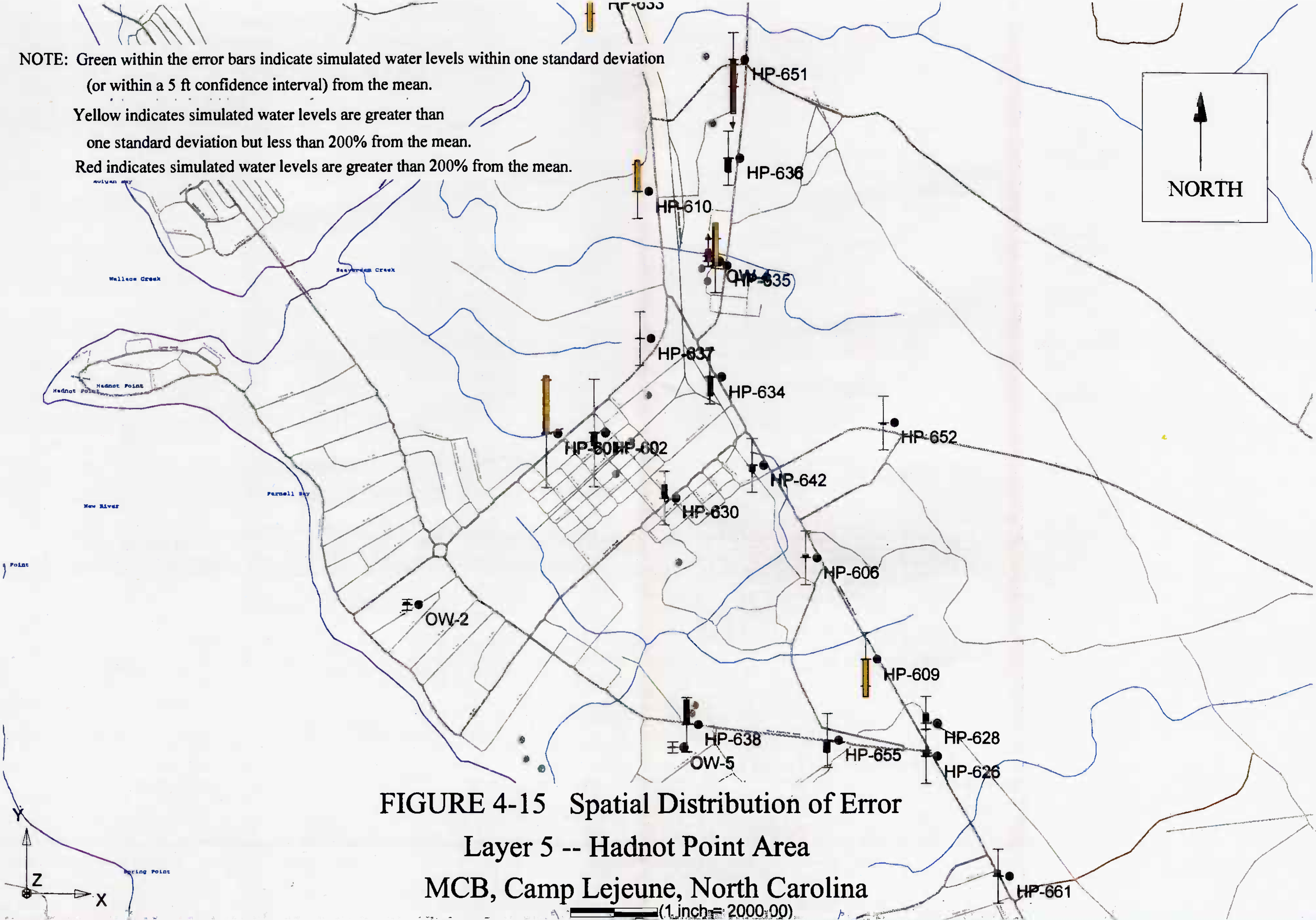


FIGURE 4-15 Spatial Distribution of Error
Layer 5 -- Hadnot Point Area
MCB, Camp Lejeune, North Carolina

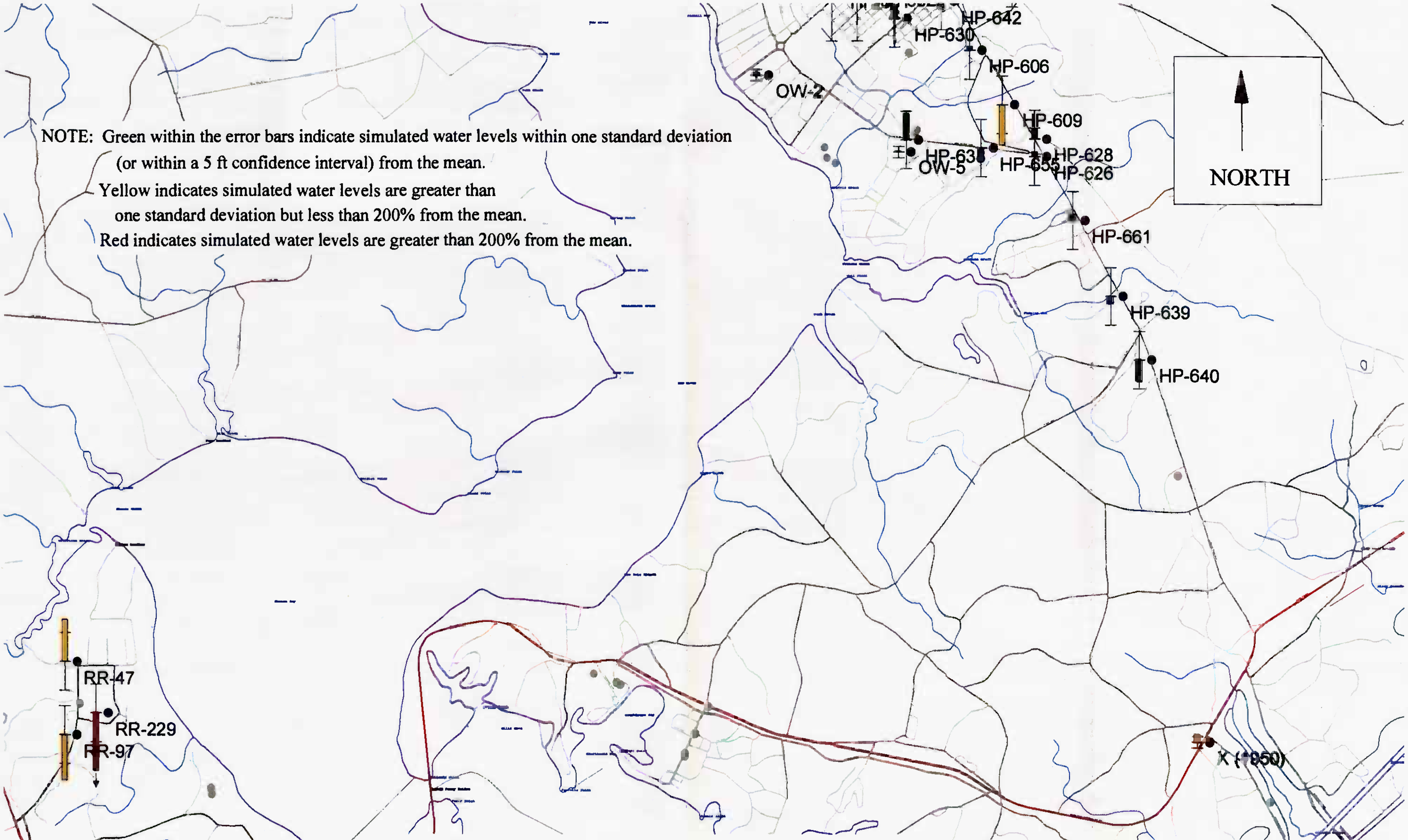


FIGURE 4-16 Spatial Distribution of Error
Layer 5 -- Southern Areas --
MCB, Camp Lejeune, North Carolina

(1 inch = 4000.00)

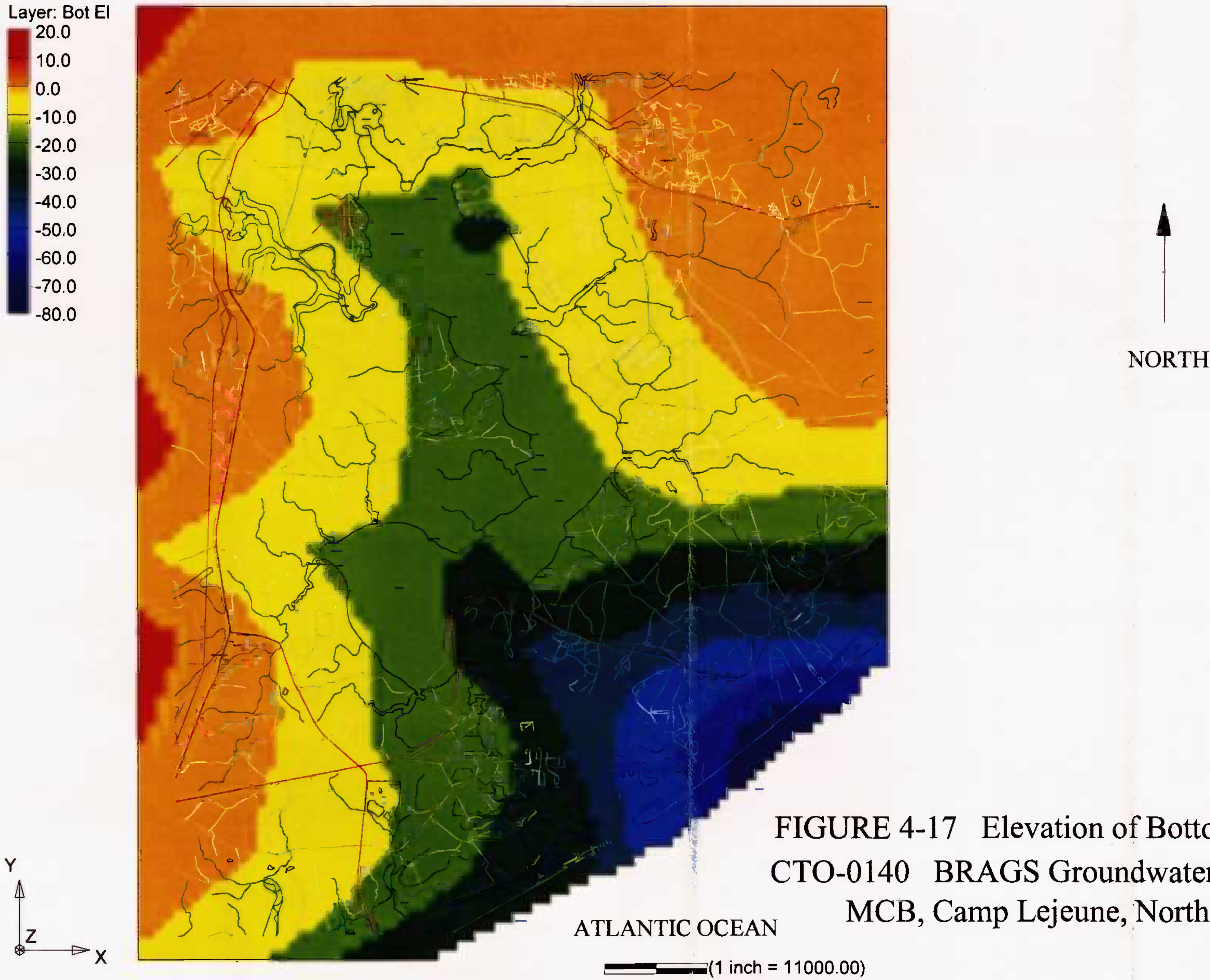


FIGURE 4-17 Elevation of Bottom of Layer 1
CTO-0140 BRAGS Groundwater Flow Model
MCB, Camp Lejeune, North Carolina

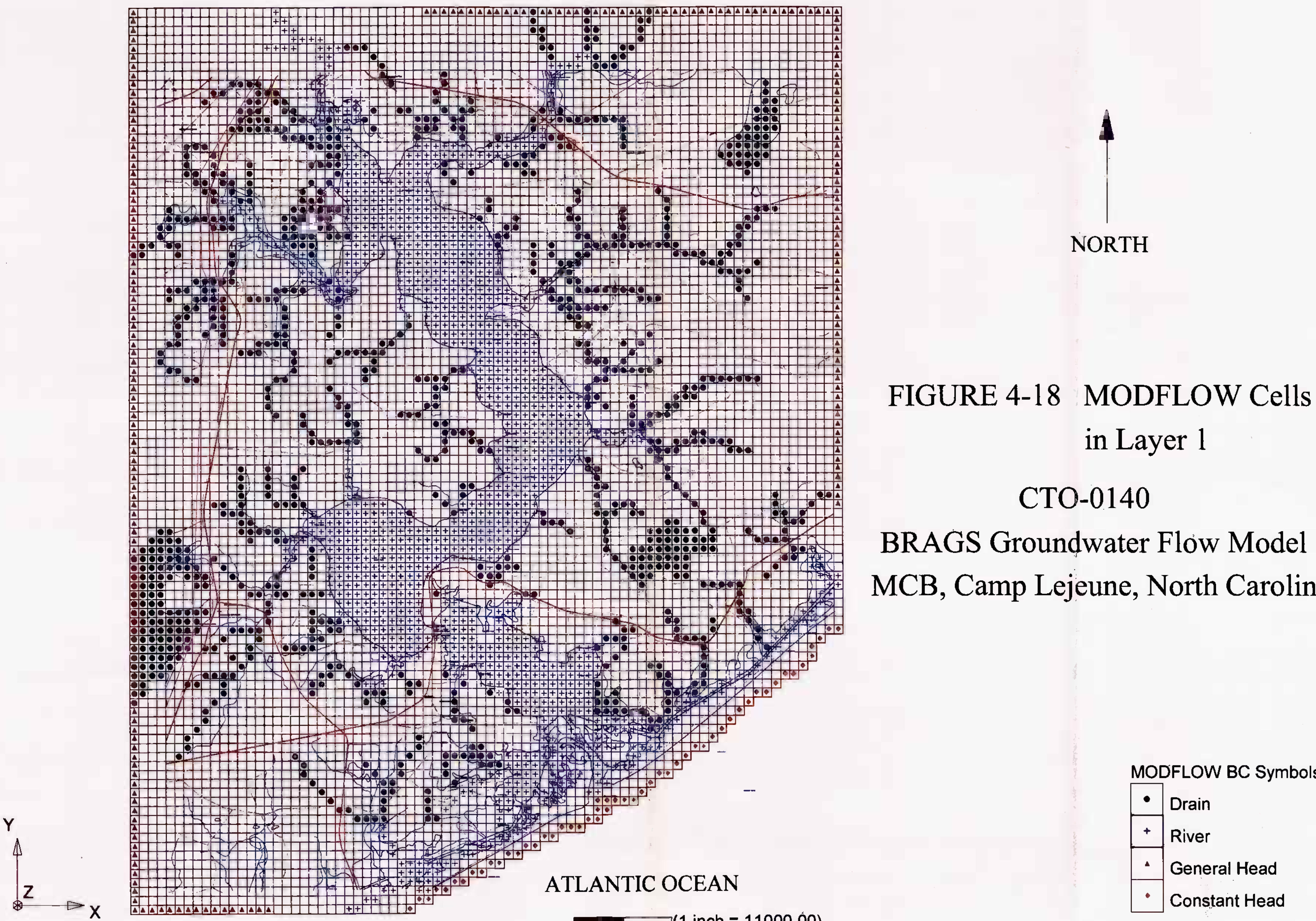


FIGURE 4-18 MODFLOW Cells in Layer 1

CTO-0140

BRAGS Groundwater Flow Model

MCB, Camp Lejeune, North Carolina

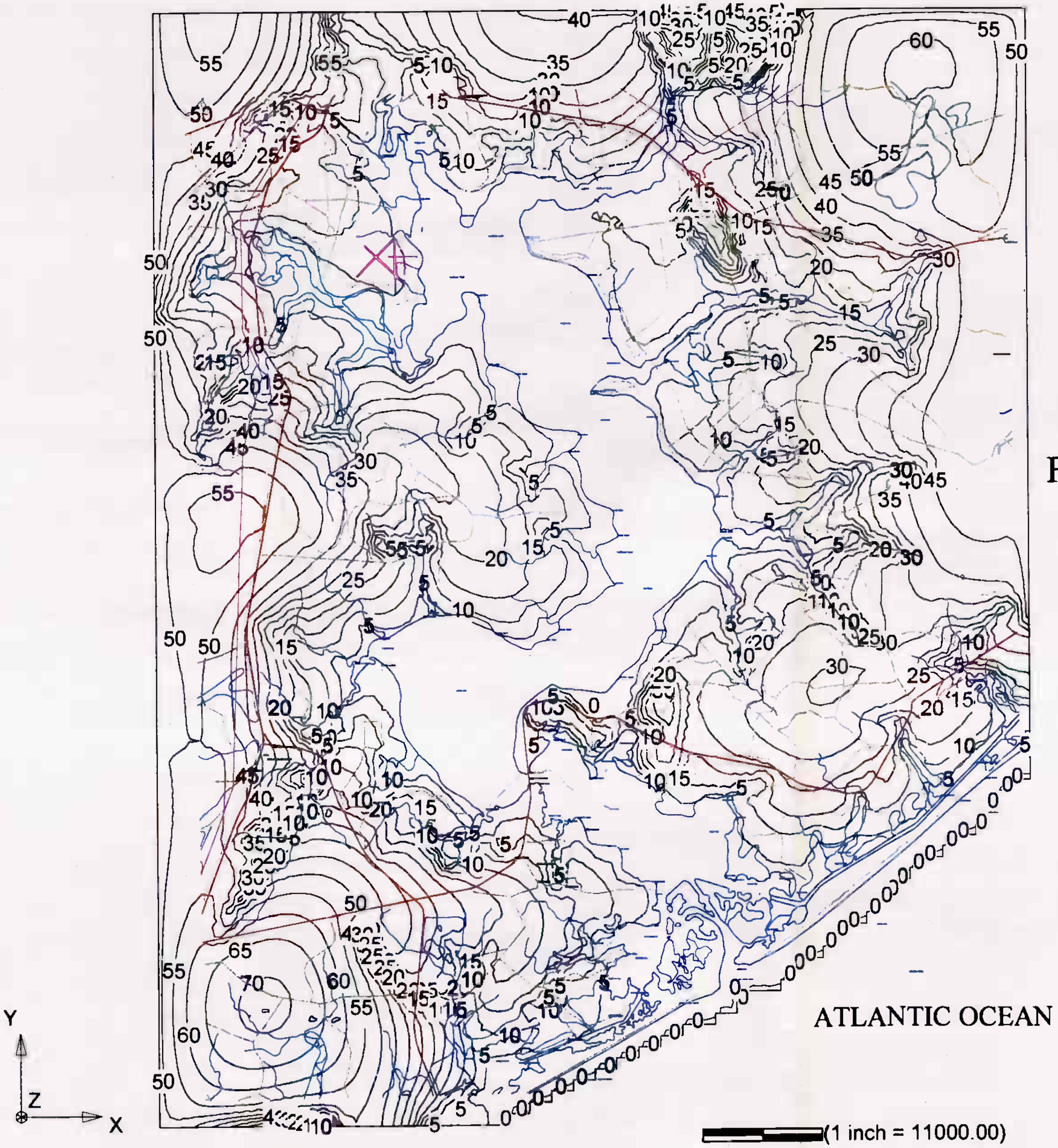


FIGURE 4-19 Simulated Water Table Elevation Contours in Layer 1

CTO-0140

BRAGS Groundwater Flow Model
MCB, Camp Lejeune, North Carolina

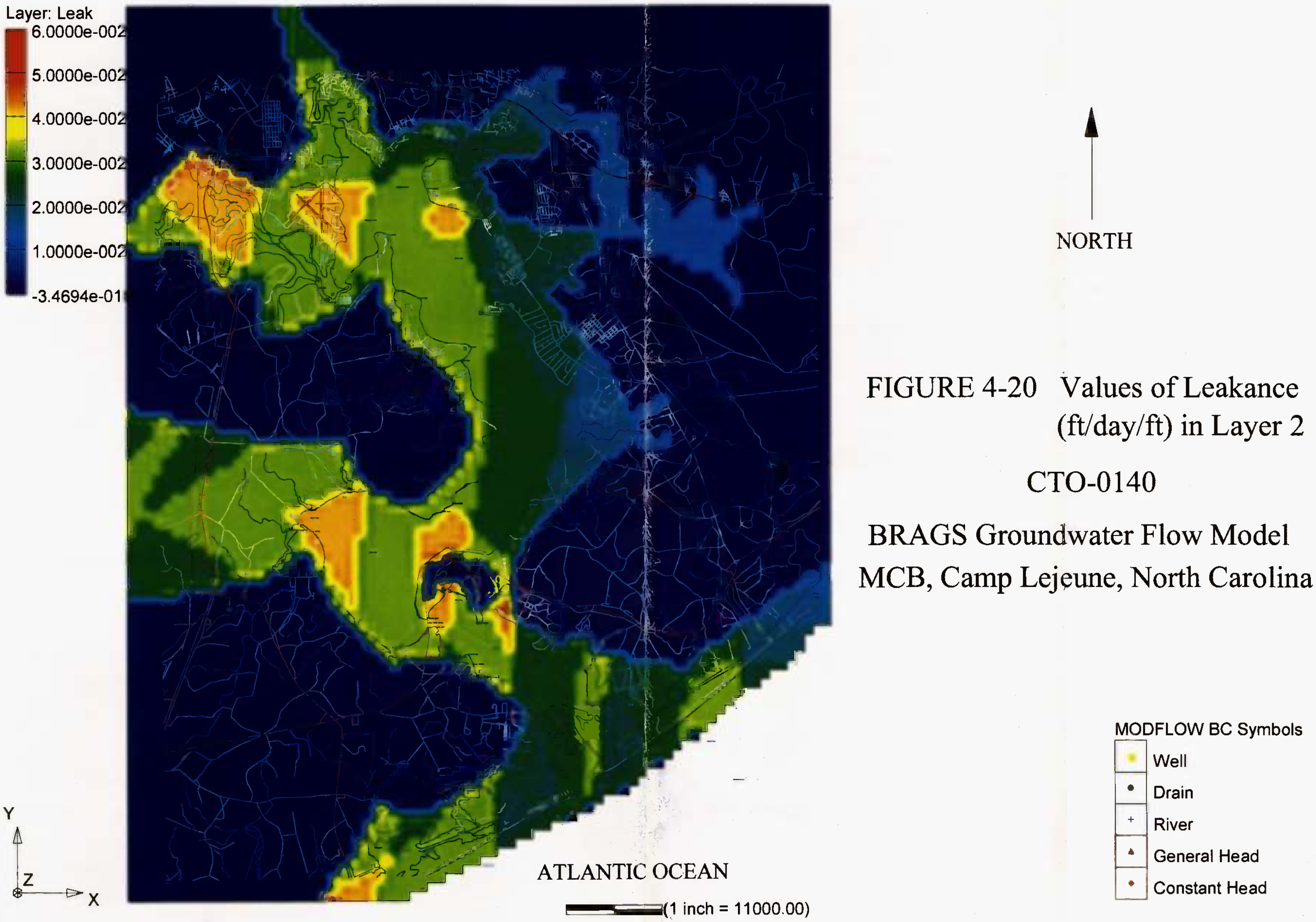


FIGURE 4-20 Values of Leakance (ft/day/ft) in Layer 2

CTO-0140

BRAGS Groundwater Flow Model
MCB, Camp Lejeune, North Carolina

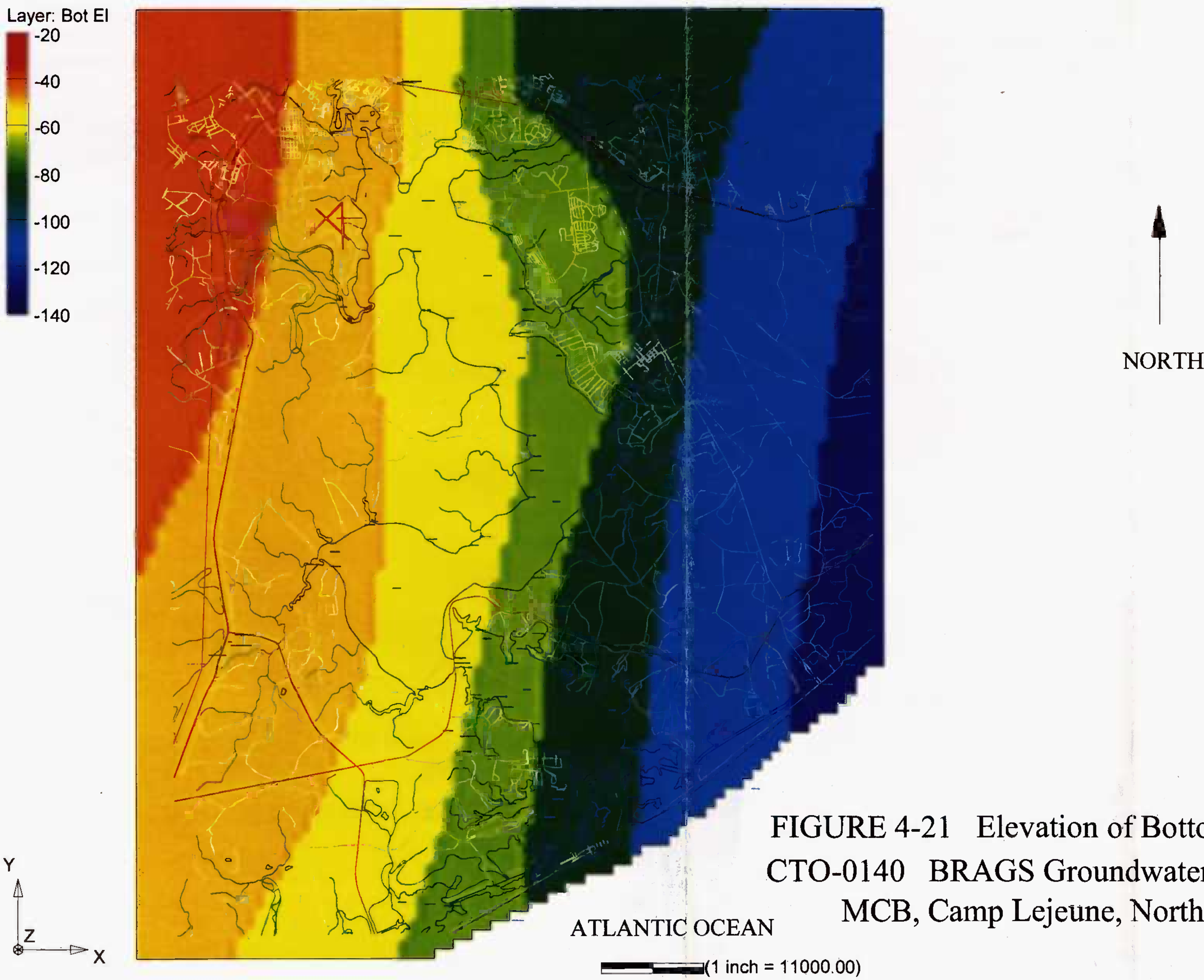


FIGURE 4-21 Elevation of Bottom of Layer 3
CTO-0140 BRAGS Groundwater Flow Model
MCB, Camp Lejeune, North Carolina

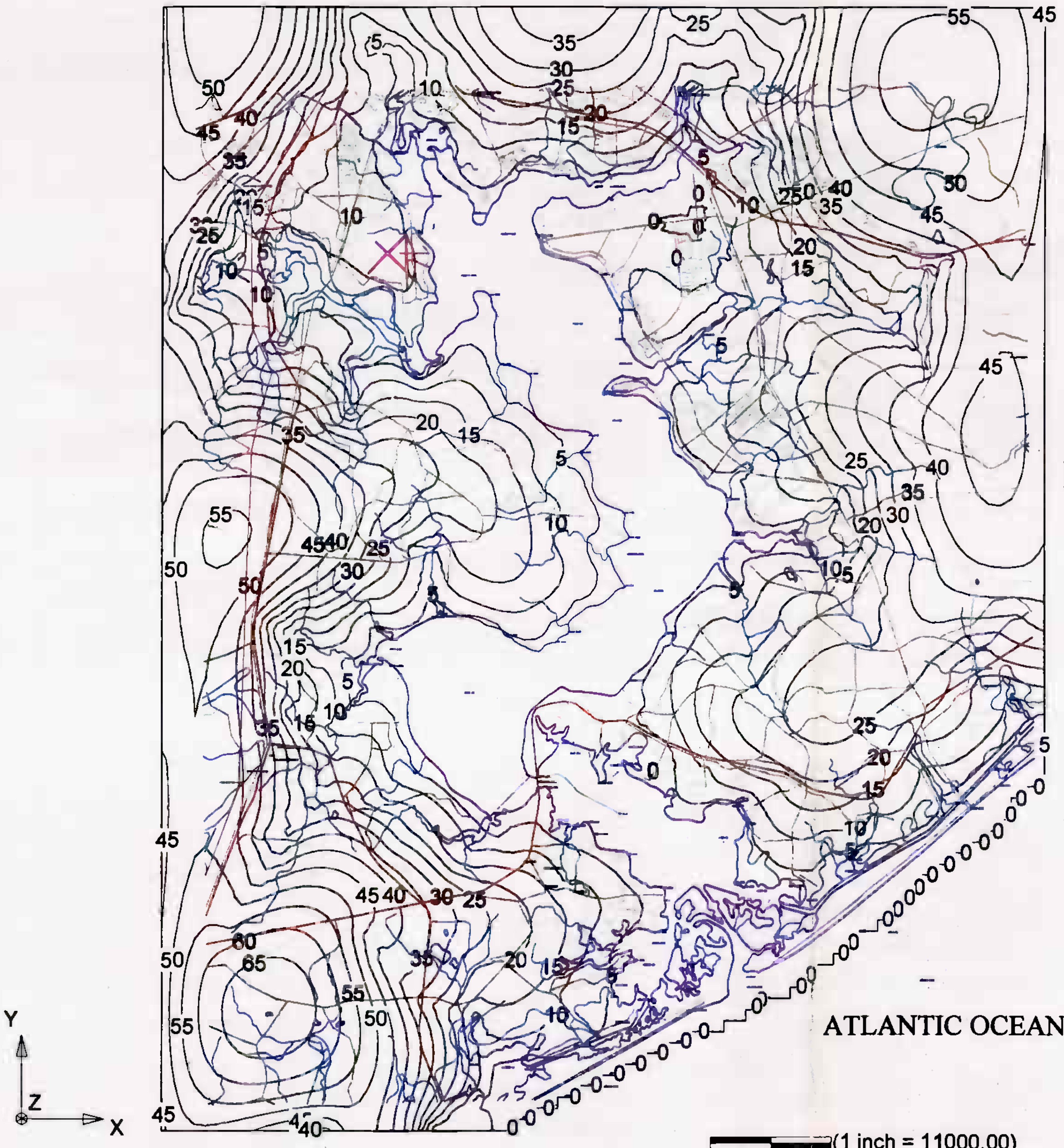


FIGURE 4-22 Simulated Groundwater Elevation Contours in Layer 3

CTO-0140

BRAGS Groundwater Flow Model
MCB, Camp Lejeune, North Carolina

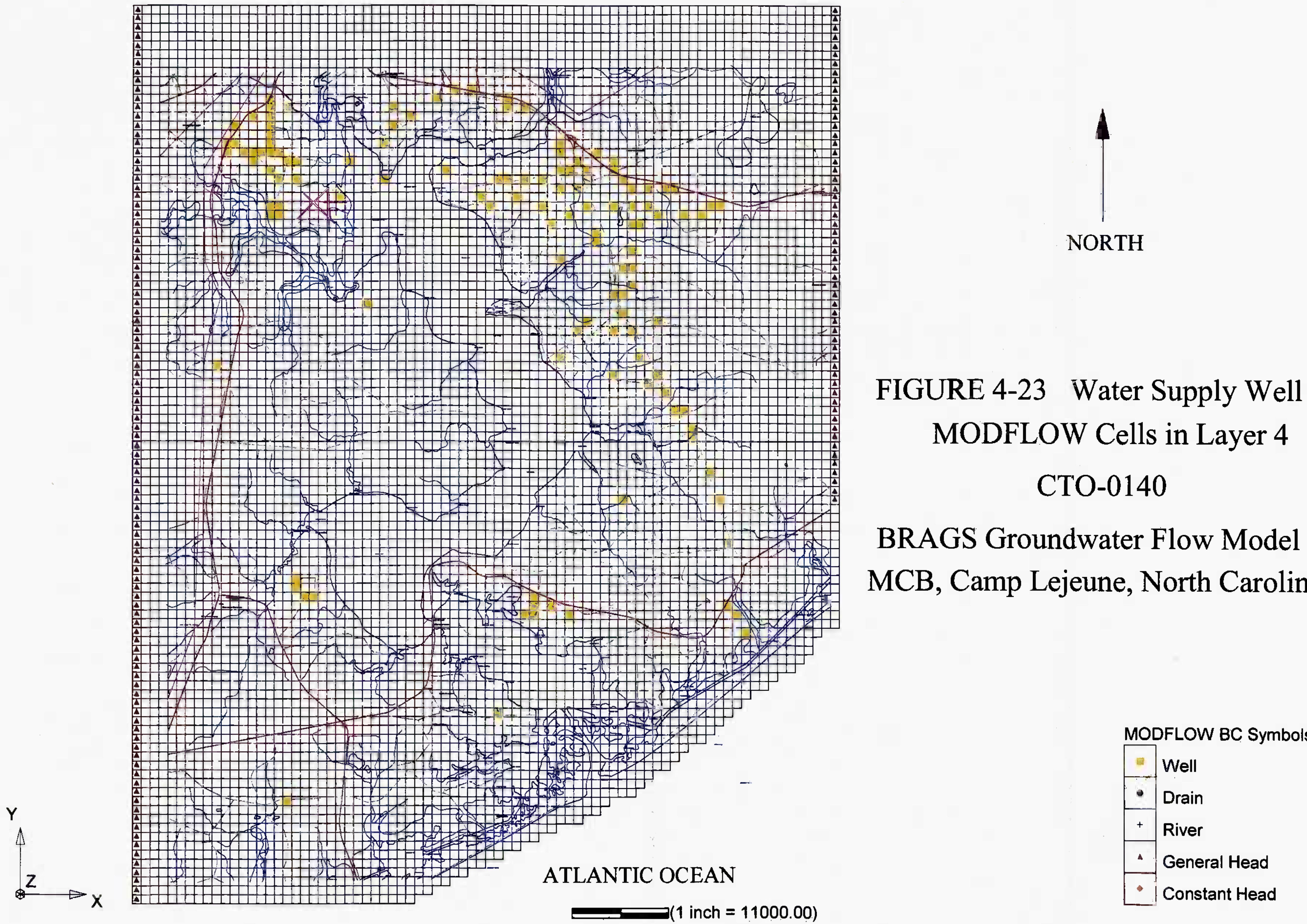


FIGURE 4-23 Water Supply Well MODFLOW Cells in Layer 4

CTO-0140

BRAGS Groundwater Flow Model
MCB, Camp Lejeune, North Carolina

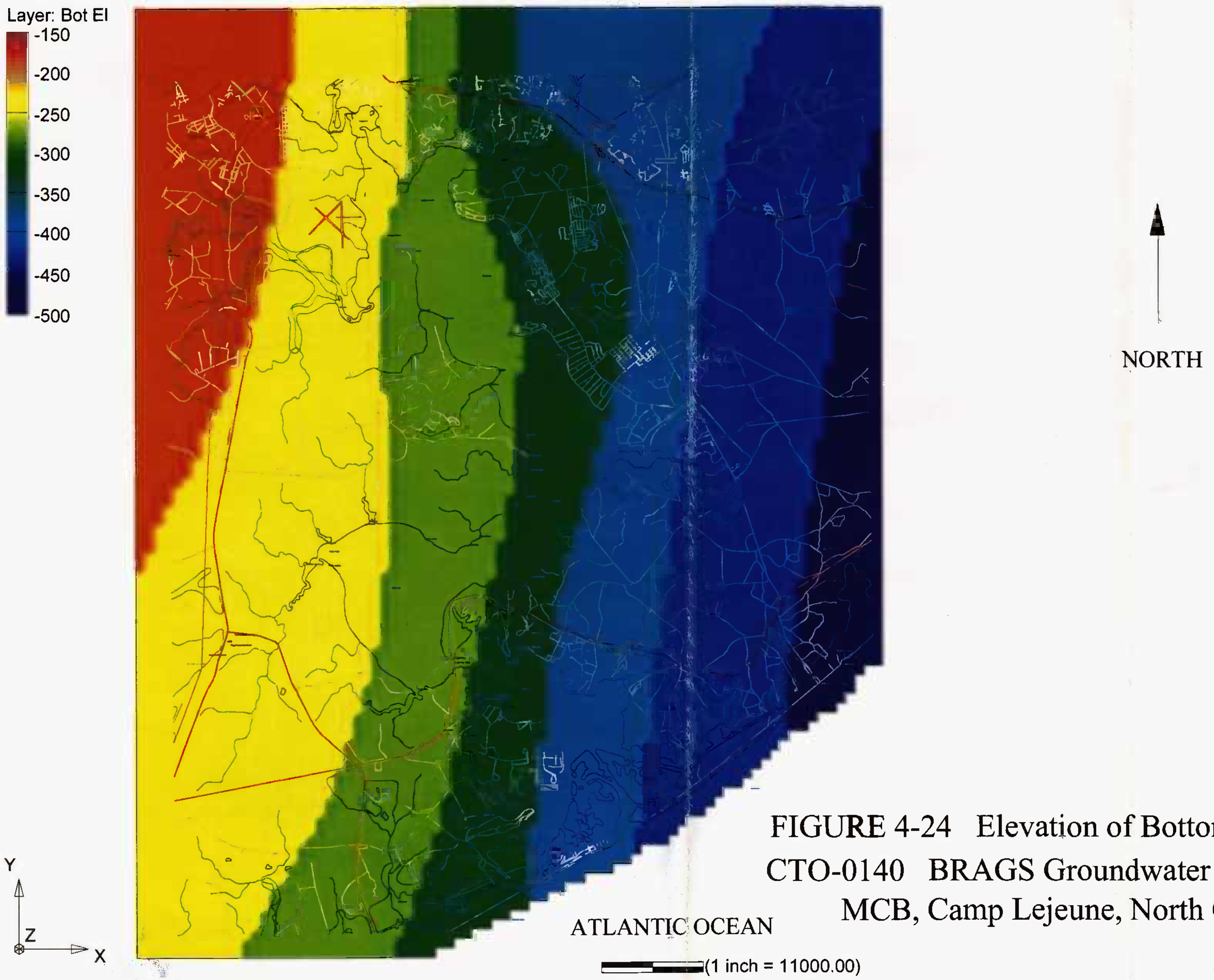
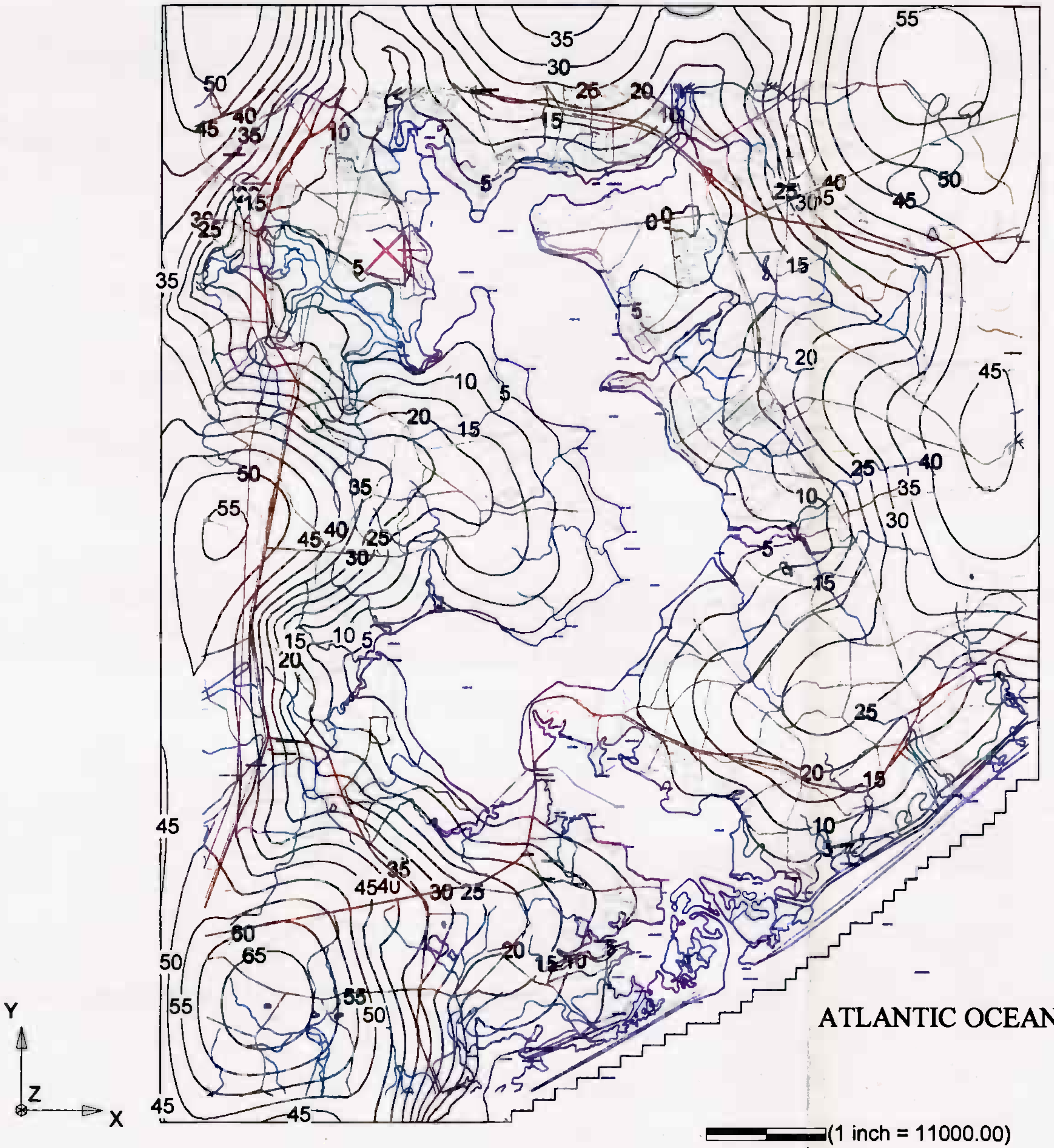


FIGURE 4-24 Elevation of Bottom of Layer 5
CTO-0140 BRAGS Groundwater Flow Model
MCB, Camp Lejeune, North Carolina



NORTH

FIGURE 4-25 Simulated Groundwater Elevation Contours in Layer 5

CTO-0140

BRAGS Groundwater Flow Model
MCB, Camp Lejeune, North Carolina

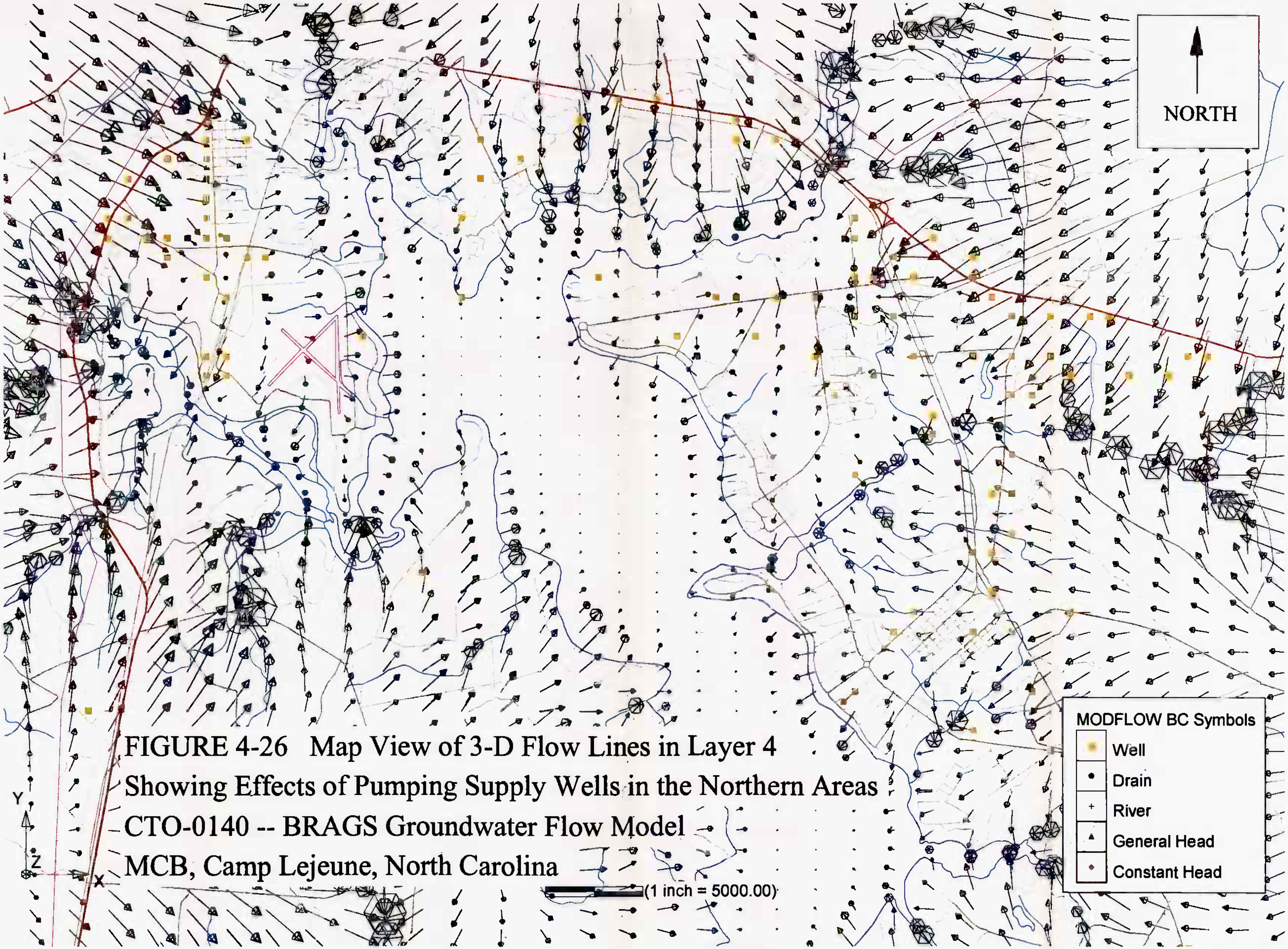


FIGURE 4-26 Map View of 3-D Flow Lines in Layer 4
Showing Effects of Pumping Supply Wells in the Northern Areas
CTO-0140 -- BRAGS Groundwater Flow Model
MCB, Camp Lejeune, North Carolina

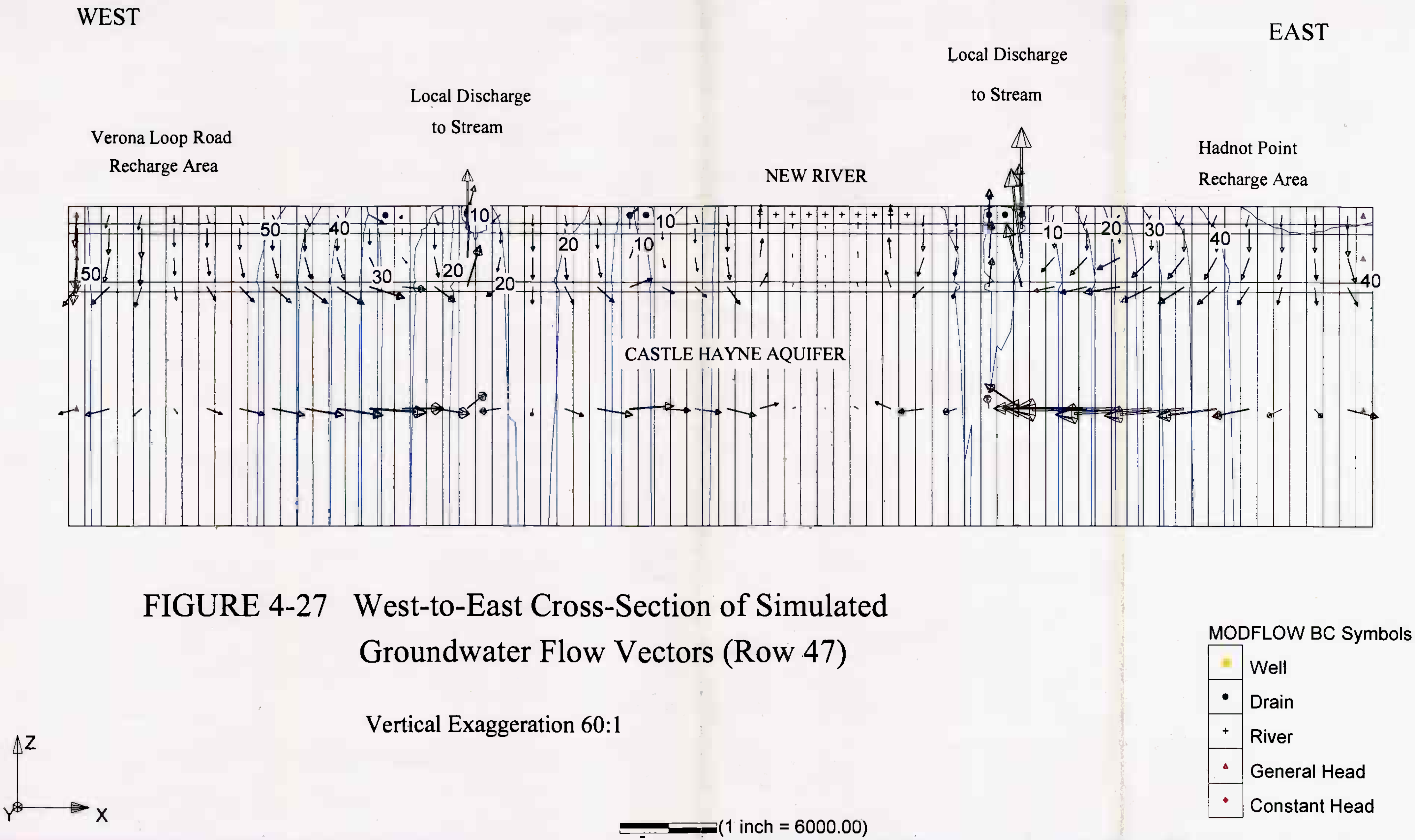


FIGURE 4-27 West-to-East Cross-Section of Simulated Groundwater Flow Vectors (Row 47)

Figure 4-28 Effects of Recharge

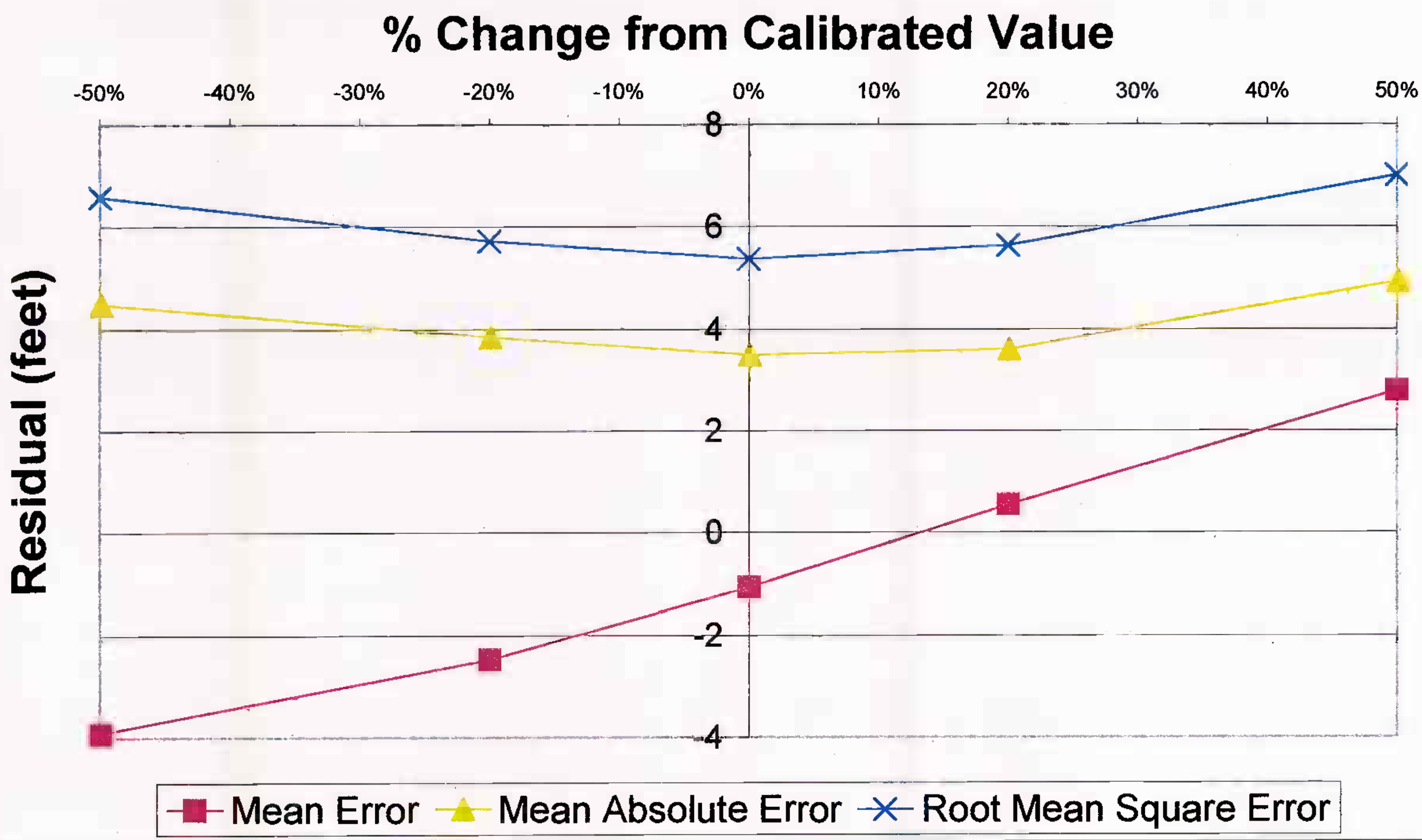
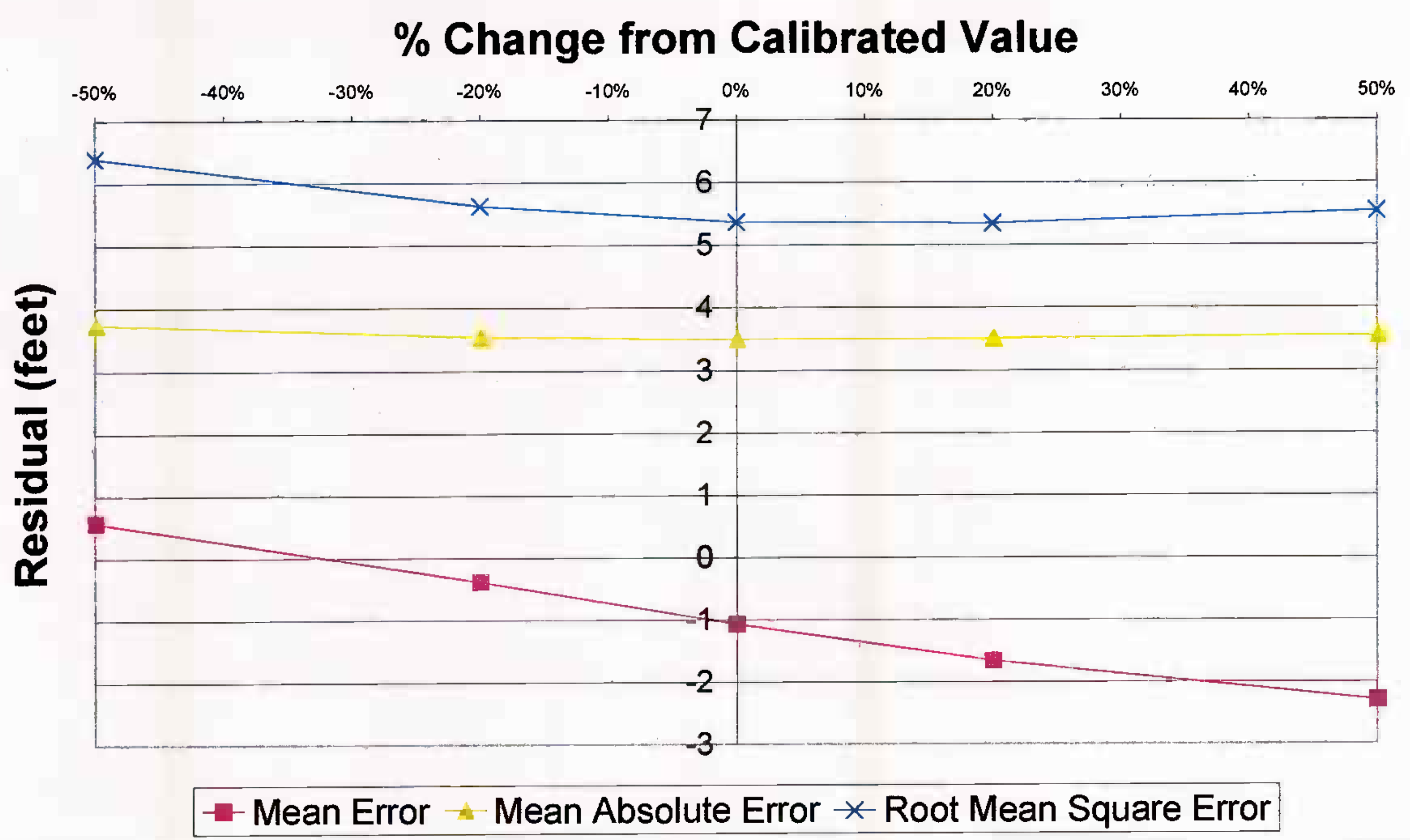


Figure 4-29 Effects of Horizontal K



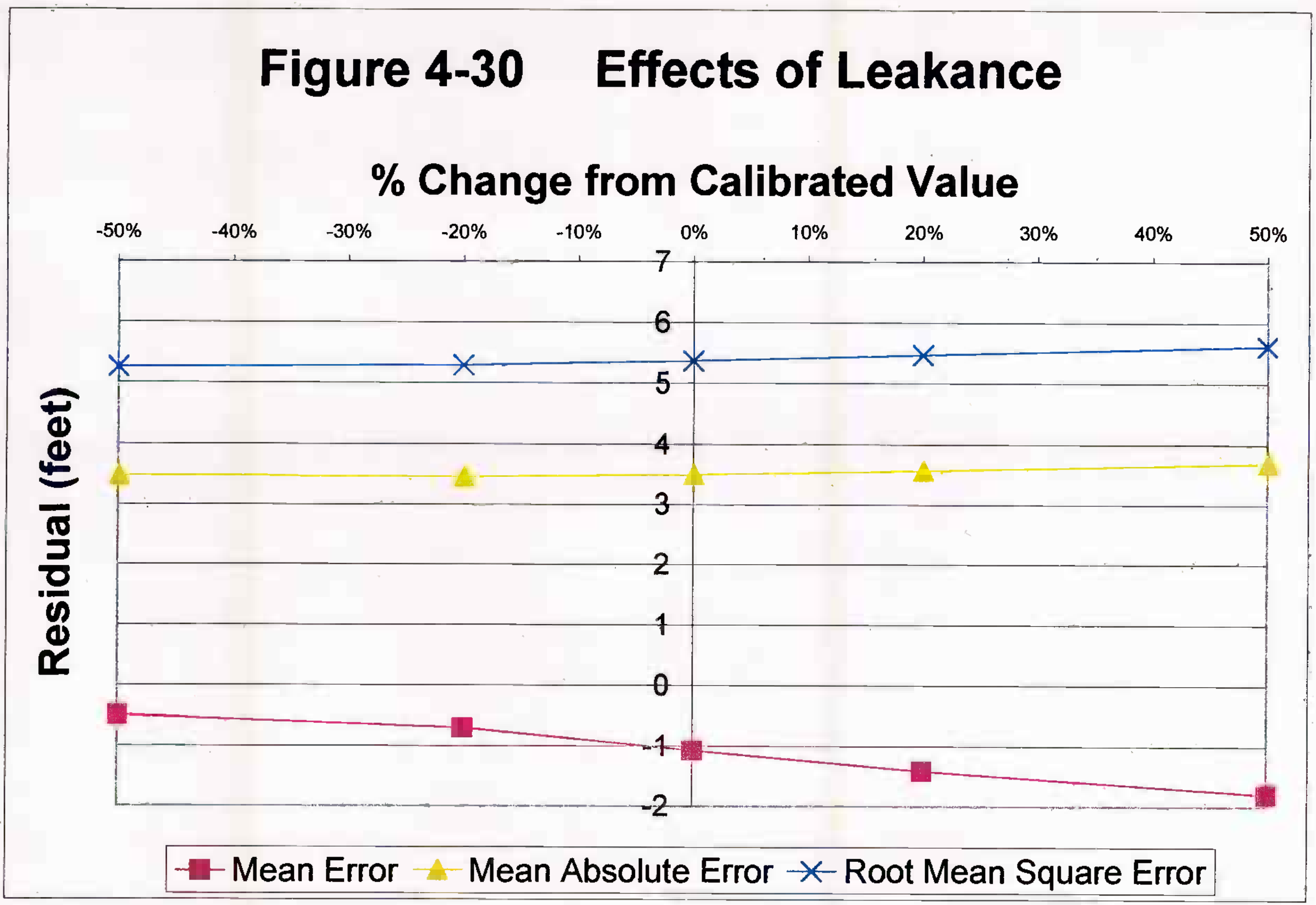


Figure 4-30 Effects of Leakance
% Change from Calibrated Value
-50%
-40%
-30%
-20%
-10%
0%
10%
20%
30%
40%
50%
7
6
5
4
3
2
1
0
-1
-2
Residual (feet)
Mean Error
Mean Absolute Error
Root Mean Square Error

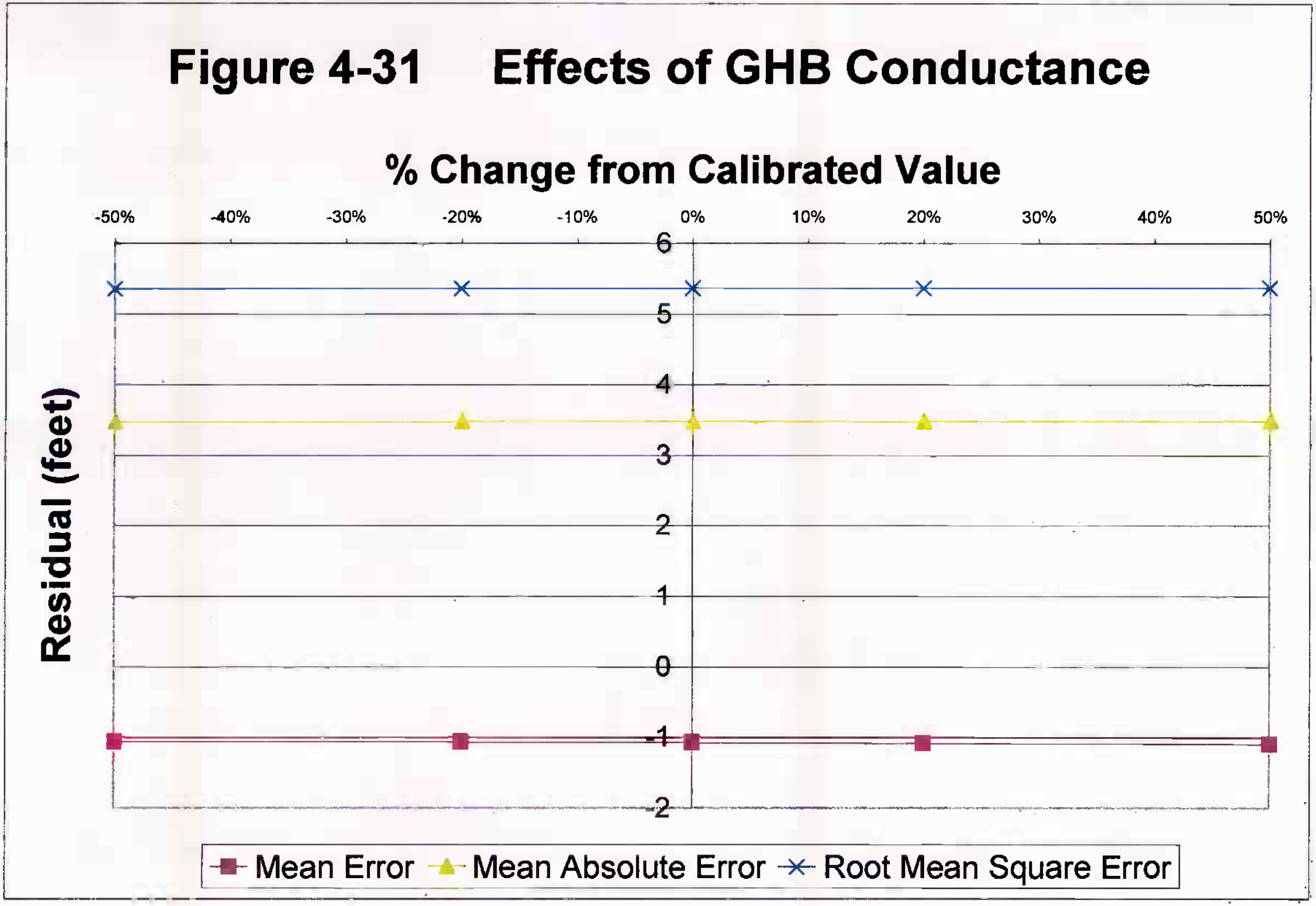
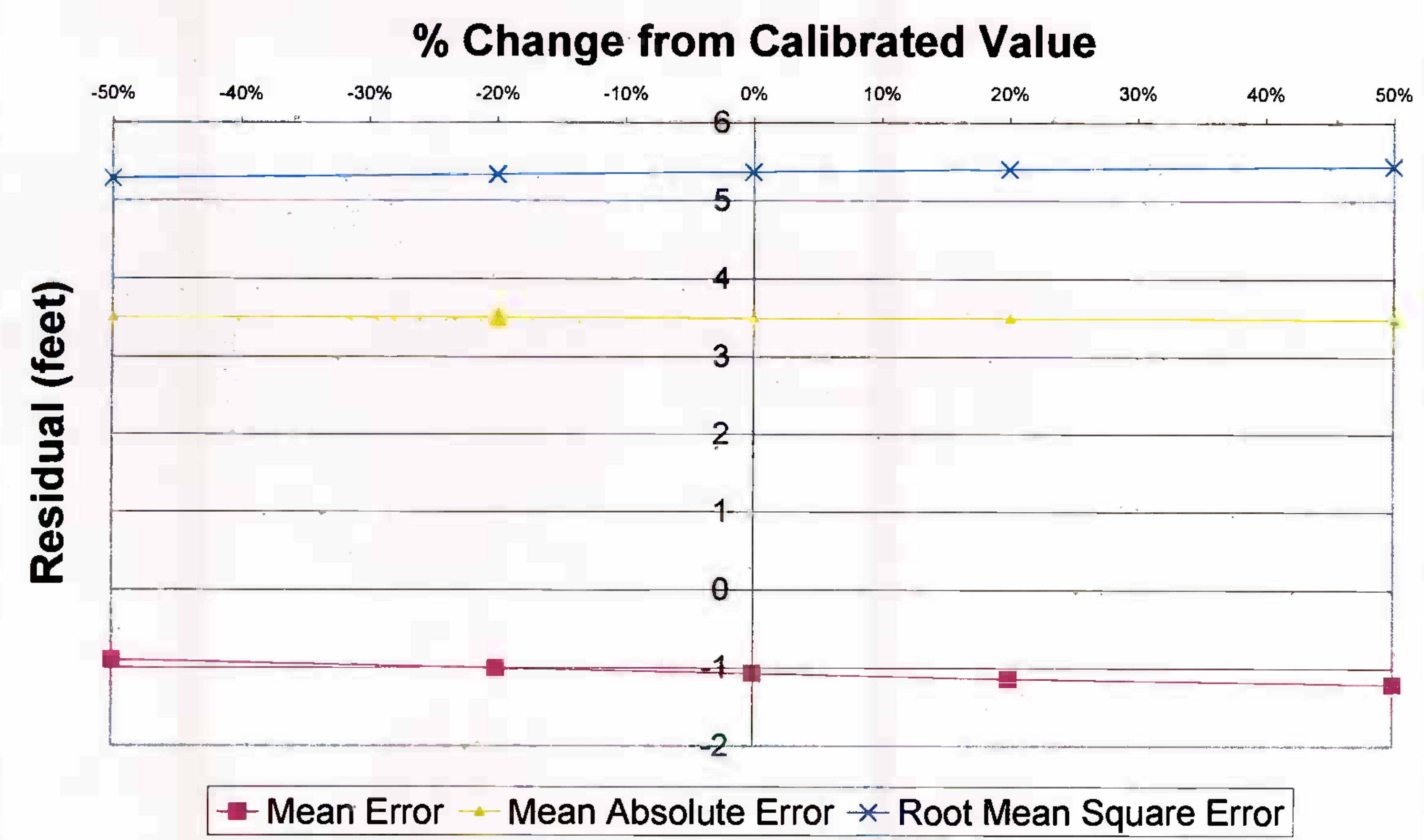


Figure 4-31 Effects of GHB Conductance
% Change from Calibrated Value
-50%
-40%
-30%
-20%
-10%
0%
10%
20%
30%
40%
50%
6
5
4
3
2
1
0
-1
-2
Residual (feet)
Mean Error
Mean Absolute Error
Root Mean Square Error

Figure 4-32 Effects of River Conductance



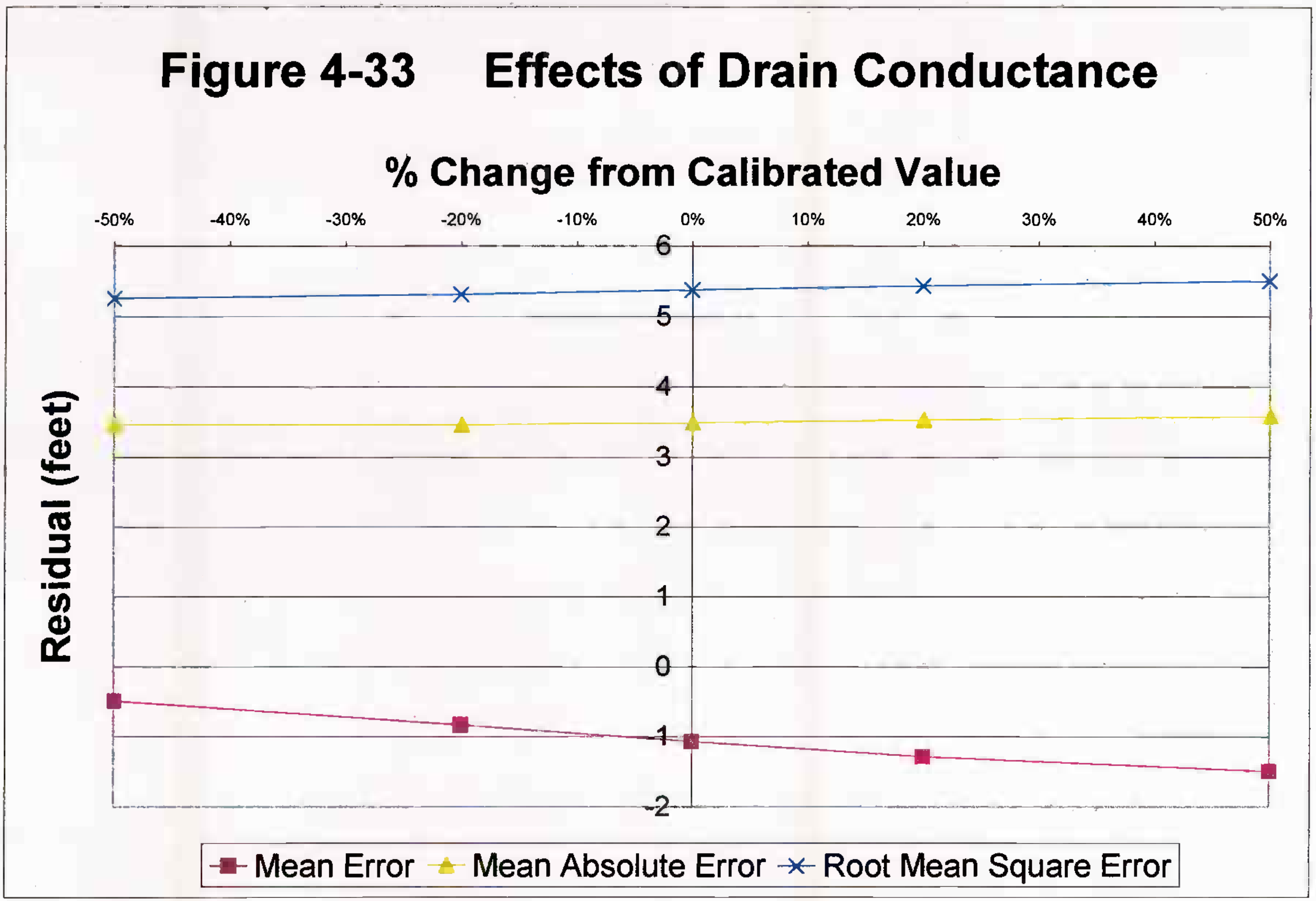


Figure 4-33 Effects of Drain Conductance
% Change from Calibrated Value
-50%
-40%
-30%
-20%
-10%
0%
10%
20%
30%
40%
50%
6
5
4
3
2
1
0
-1
-2
Residual (feet)
Mean Error
Mean Absolute Error
Root Mean Square Error

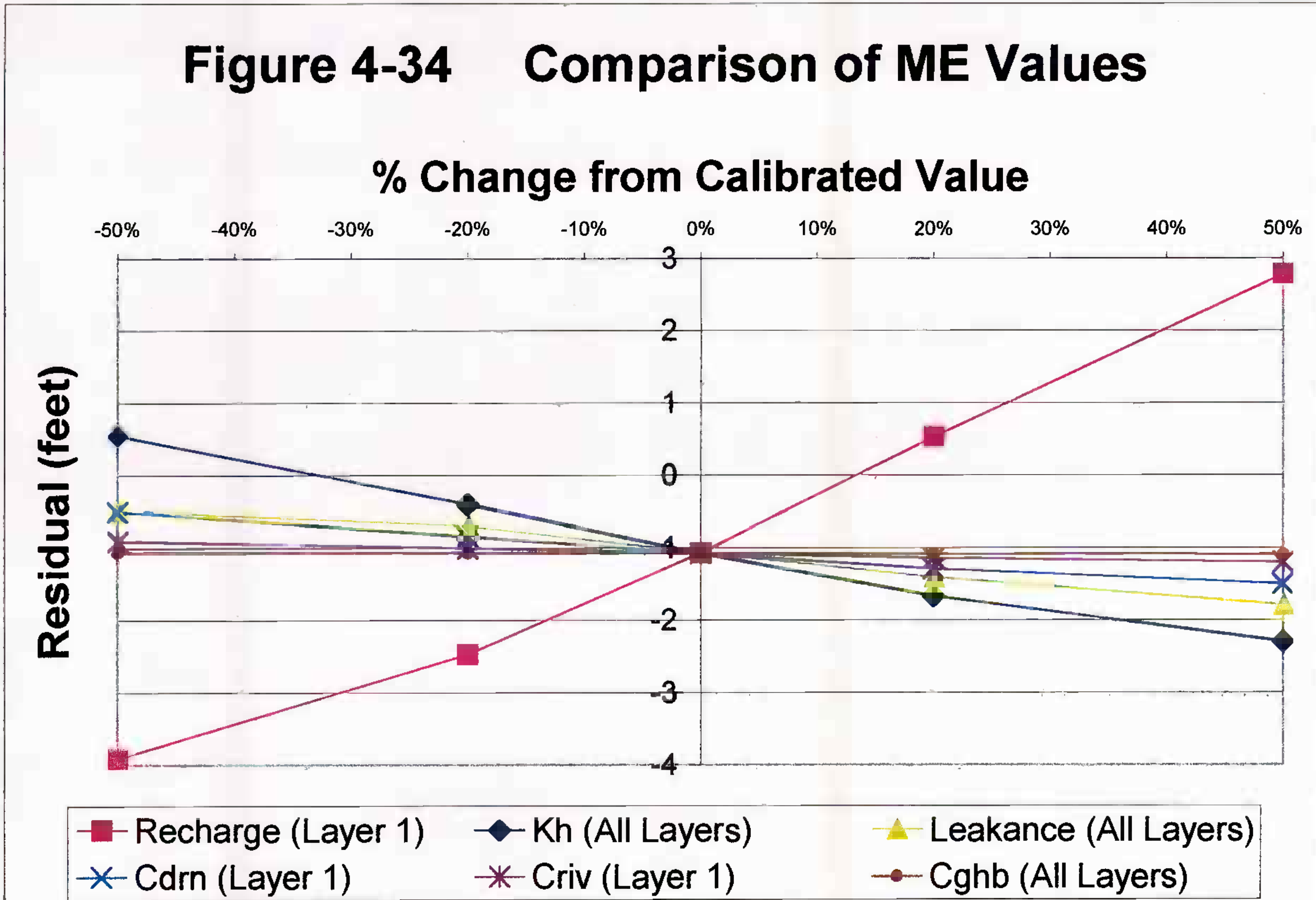


Figure 4-34 Comparison of ME Values
% Change from Calibrated Value
-50%
-40%
-30%
-20%
-10%
0%
10%
20%
30%
40%
50%
3
2
1
0
-1
-2
-3
-4
Residual (feet)
Recharge (Layer 1)
Kh (All Layers)
Leakance (All Layers)
Cdrn (Layer 1)
Criv (Layer 1)
Cghb (All Layers)

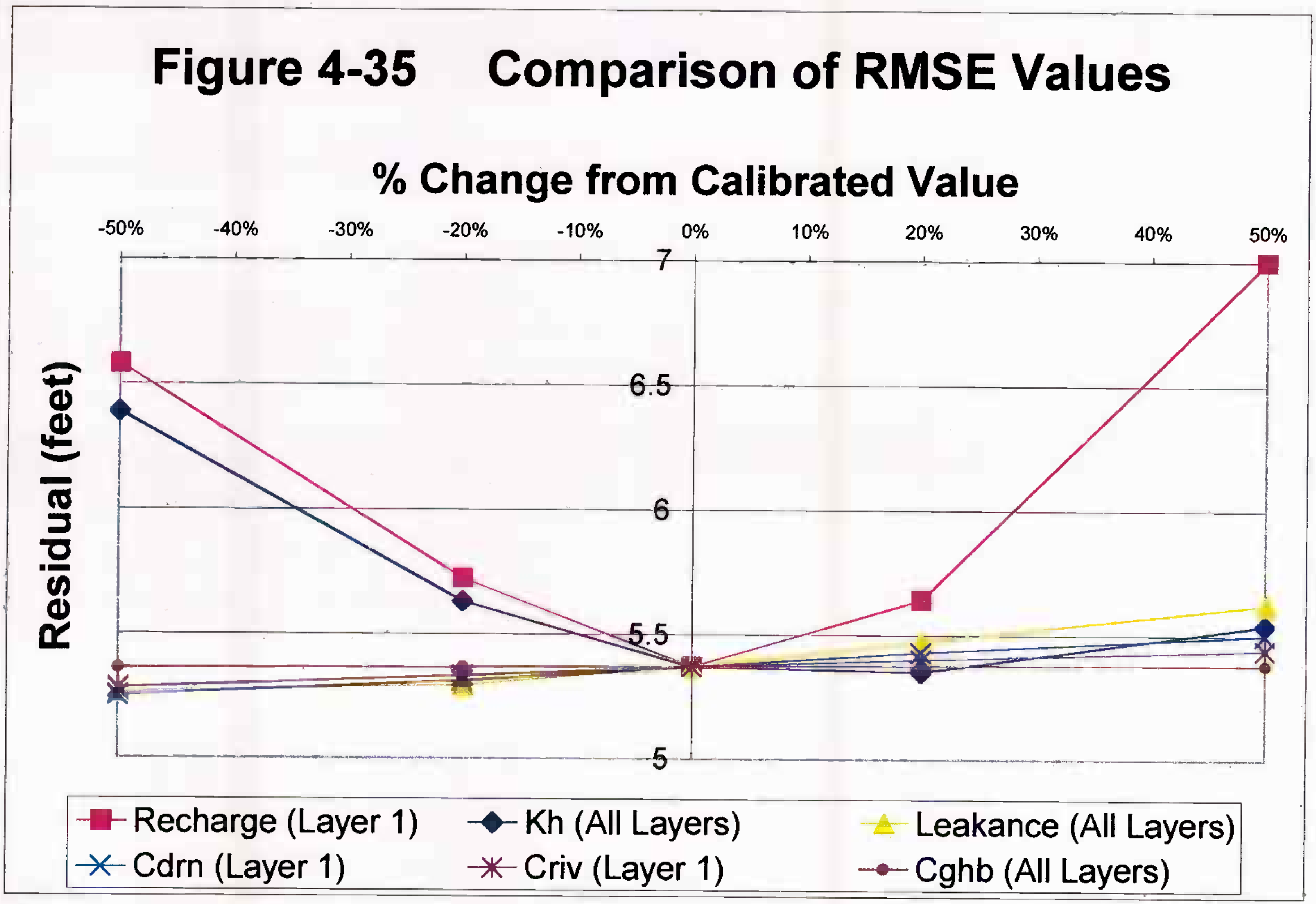


Figure 4-35 Comparison of RMSE Values
% Change from Calibrated Value
-50%
-40%
-30%
-20%
-10%
0%
10%
20%
30%
40%
50%
7
6.5
6
5.5
5
Residual (feet)
Recharge (Layer 1)
Kh (All Layers)
Leakance (All Layers)
Cdrn (Layer 1)
Criv (Layer 1)
Cghb (All Layers)

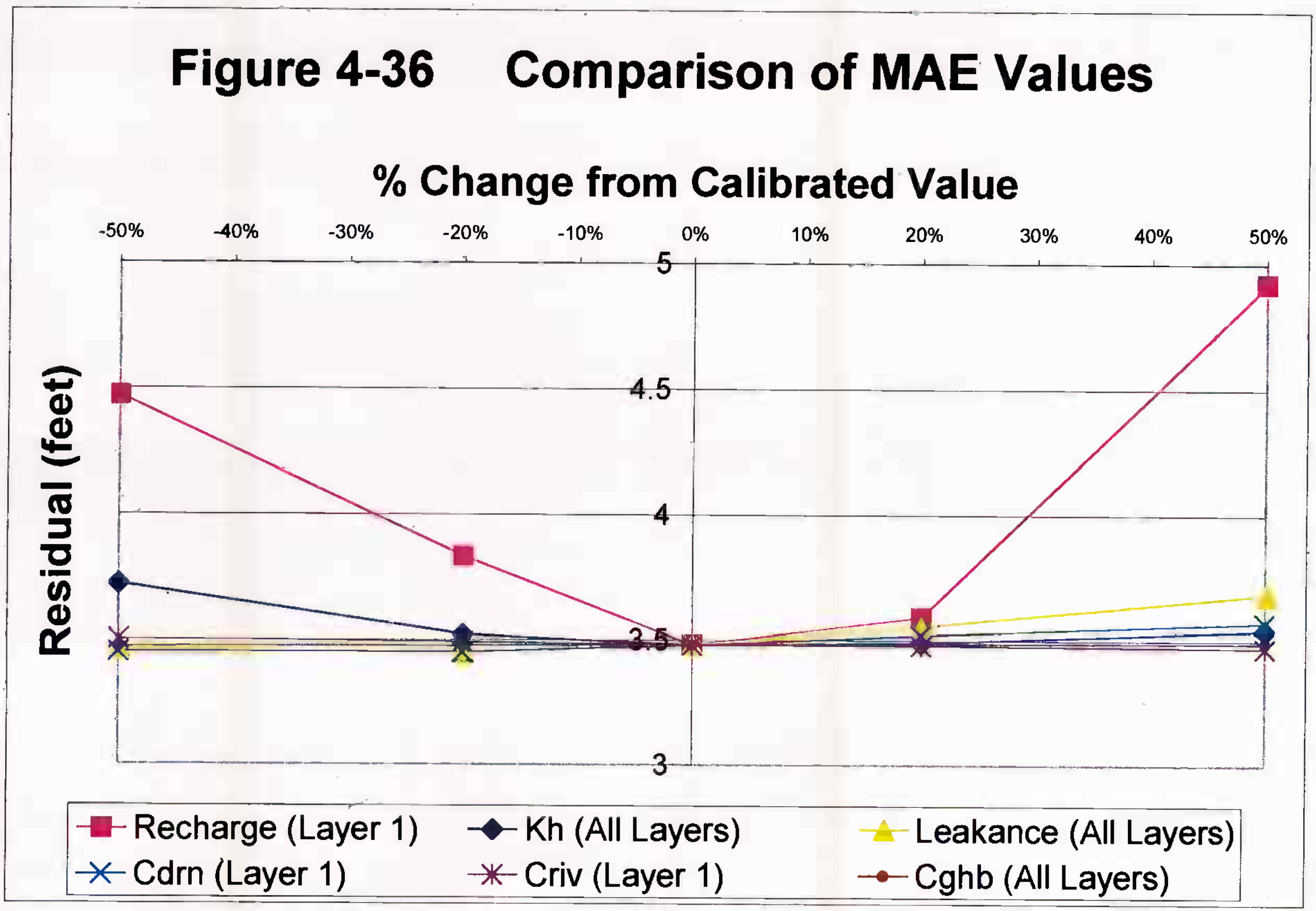


Figure 4-36 Comparison of MAE Values
% Change from Calibrated Value
-50%
-40%
-30%
-20%
-10%
0%
10%
20%
30%
40%
50%
5
4.5
4
3.5
3
Residual (feet)
Recharge (Layer 1)
Kh (All Layers)
Leakance (All Layers)
Cdrn (Layer 1)
Criv (Layer 1)
Cghb (All Layers)

5.0 SITE 82 GROUNDWATER FLOW MODEL

Site 82, the Piney Green VOC Area, is a forested area approximately 30 acres in size located on the southern boundary of Wallace Creek and north of the Hadnot Point Industrial Area (see Figure 5-1). Disposal of chlorinated solvents and petroleum products occurred at the site during the 1950's through the 1970's when the adjacent site (Site 6) was used for an open lot storage area (referred to as Storage Lot 203) for various industrial materials and supplies used by the Base. Studies conducted at the site (NUS, 1991; Baker, 1992 through 1997) indicated high levels (as high as 97,000 ug/L) of the chlorinated solvent trichloroethene (TCE) in the Castle Hayne Aquifer. Consequently, a groundwater pump and treatment system was designed for the site to address the volatile organics in the underlying aquifers and to mitigate possible migration to nearby water supply wells (notably HP-633).

After the BRAGS model for Camp Lejeune was completed, the model for Site 82 was started. Site 82 is immediately adjacent and due north of Site 6. The first step was to use the BRAGS model to get a first-order approximation of the simulated groundwater conditions near Site 82. The external boundary conditions used in the Site 82 model were derived from the simulated heads in the BRAGS model.

Flow directions were modeled using MODPATH. MODPATH is a particle-tracking code also developed by the USGS (Pollock, 1989) that uses the results of MODFLOW to generate particle traces (or pathlines) that result from groundwater advection (flow) only. Although no dispersion, reaction, and degradation of particles are possible with this type of software, it is most useful to generate capture zones around individual wells to demonstrate contaminant capture. Electronic model input and output for the Site 82 model can be found on CD-ROM in Appendix C.

5.1 Finite-Difference Layered Grid

The finite-difference grid superimposed over Site 82 had variable spacing: square and rectangular cells range from 25 to 1,000 feet in length (see Figure 5-2). The grid was comprised of 72 rows (about 10,400 feet north to south) and 94 columns (about 13,600 feet east to west) over an area of approximately five square miles.

The Site 82 model consisted of only two layers: the top layer representing the surficial unit and the bottom layer the Castle Hayne Aquifer. The Site 82 model was "quasi-3d" meaning that the confining layers were represented not by actual low hydraulic conductivity layers, but by a "leakance factor" used by MODFLOW to calculate leakance between layers. Because the average thickness of the Castle Hayne confining unit near Camp Lejeune is about 10 feet, this was the thickness of the pseudo-confining layer. That is, the bottom of layer 1 is 10 feet above the top of layer 2.

5.2 Model Boundary Conditions

Boundaries in MODFLOW include external and internal boundaries. External boundaries can include specified head or general head boundary cells. Internal boundaries include well, river, stream, and drain cells. For the Site 82, no specified head or stream cells were used. External boundaries were general head boundary cells and internal boundaries were well, river and drain cells.

5.2.1 General Head Boundary Cells

General head boundary cells are head-dependant flow cells that allow flow into or out of the cell depending on two things: 1) the head differential between the assigned value and that in the surrounding aquifer and, 2) an assigned constant of proportionality. In the Site 82 model, the assigned value of head represents a head value (e.g., of a surface water body) at some distance beyond the model boundary and the proportionality constant represents the hydraulic conductivity of the aquifer between the model boundary and the surface water body.

Both layers had general head boundary cells placed along the outer boundaries to simulate the ambient groundwater gradient (as determined by the BRAGS model). The values of head assigned to each cell were chosen to represent the gradient of regional groundwater flow. The proportionality constants were adjusted by trial and error during the calibration process until a reasonable fit was achieved at the boundaries.

5.2.2 Well Cells

Wells cells are specified (constant) flux boundaries which keep a constant flow rate throughout the specified time period. Positive values recharge to groundwater and negative values discharge from groundwater. These cells were placed at the locations of the water supply wells and at the locations of the existing and proposed extraction wells. As in the BRAGS model, the wells were assigned average daily pumping rates in cubic feet per day (negative to represent groundwater discharge). All available well locations were plotted even if they were turned off. This will help in the future if they are turned on again.

The state planar coordinates of the water supply wells were converted from the latitude and longitude as recorded in Cardinell et al (1993). The state planar coordinates of the monitoring and extraction wells were taken from Site 82 survey data.

5.2.3 River Cells

River cells are head-dependant flow cells in which the elevations of the surface water and river bottom are held constant (at surveyed or mapped elevations) and the thickness and conductance of the sediments control the flow rate of water to or from the cell. If the stream or pond level is higher than the surrounding groundwater, the river cell allows water to recharge the groundwater. Conversely, if the water level in the stream or pond is lower than the groundwater, the groundwater discharges to the surface water body. The equation for river conductance C_R was given in the previous chapter.

River cells were used to represent Wallace Creek near Site 82 where its elevation is mean sea level. This includes the estuary portions of Wallace Creek (as defined on the site maps).

5.2.4 Drain Cells

Drain cells function similarly to river cells except that they cannot recharge the groundwater when the ambient water table drops below the drain elevation. Streams and swamps were represented by drain cells because it was reasonably assumed that they only receive groundwater discharge and were not recharging groundwater. The elevations of the drain cells were the approximate elevations of the streams as determined by topographic mapping of the area.

5.3 Steady-State Modeling Process

Like the BRAGS model, the Site 82 model was steady-state and all values of drawdown are assumed to have reached equilibrium. This assumption is valid when applied over the long term (years or decades) to understand how groundwater flows within the modeled system. Again, the most important assumption of this approach is that the diurnal pumping schedule of the water supply wells has been averaged as if pumping were a continuous event.

In the Site 82 model, new wells were introduced to the system. Therefore, it was necessary to calibrate the model to pre-pumping conditions (in the Site 82 extraction wells) before they could be "turned on."

5.3.1 Pre-Pumping Calibration Targets

Pre-pumping water elevations were measured at the monitoring wells at Sites 82 and Site 6 (Storage Lot 201 -- adjacent to Site 82) during 1992 and 1993. Table 5-1 shows that there were a total of 59 head targets: 33 shallow wells in layer 1 and 26 deep, intermediate, or supply wells in layer 2. Because of the data quality limitations of the water supply well data (discussed in the previous chapter), more credence was given to the data collected from the Site 82/Site 6 wells.

5.3.2 Calibration Methods

As in the BRAGS model, the calibration process used both "trial and error" and "parameter estimation" methods. The "parameter estimation" calibration was used generally at the beginning and at the end of the calibration process with the "trial and error" process used in the middle.

5.3.3 Statistical Evaluation of Calibration

The same statistics were used to calibrate the pre-pumping Site 82 model as were used in the BRAGS model: RM, ARM, RSD, and RMS. Generally, the degree of fit was deemed acceptable when the errors were within 10 feet of the target averages in both layers 1 and 2.

5.4 <u>Calibrated Results of Pre-Pumping Simulation</u>

This section describes the inputs and outputs of the calibrated pre-pumping simulation. Unless otherwise indicated, the input values used were taken directly from the values discussed in the previous sections.

Table 5-1 presents the errors of each target in the Site 82 groundwater flow model. The end of the table shows the statistics for each layer as well as for the entire model. All simulated heads were within 14 feet of the target values. The RM for all three layers in the Site 82 model was 1.72 feet, which means that the average of all the simulated water levels in both layers was 1.72 lower than the measured water levels. The ARM for both layers was 3.23 feet. The RSD was 3.99 feet, and the RMS was 4.35 for the final calibration.

5.4.1 Layer 1 -- Surficial Unit

5.4.1.1 Pre-Pumping Input

As in the BRAGS model, a uniform value of recharge of 11 inches per year was used in this model. Recharge occurred only in layer 1.

The top elevation of layer 1 was assigned an arbitrary uniform value of +80 feet msl. This value is greater than the water table elevation which ensures that layer 1 remains unconfined. The bottom elevations in layer 1 ranged from -20 feet to -10 feet msl (see Figure 5-3).

General head boundary cells were used to simulate the ambient groundwater at the external boundaries of the Site 82 model. On the upgradient (north and east) sides of the grid, head values of either +20 or +28 feet msl and proportionality constants of 10,000 and 5,000, respectively, were assigned to various areas of general head boundary cells (see Figure 5-4). This method achieved a reasonable match of the "incoming" water elevations. On the downgradient (west) side of the grid, head values of zero (sea level) and a proportionality constant of 50 or 5,000 were used to approximate the New River beyond the western grid boundary.

River cells were used to represent Wallace Creek near Site 82 where its elevation is mean sea level see (Figure 5-3). Using the input value of 5000 ft²/d for C_{riv} is reasonable assuming the following parameter values for the Wallace Creek:

L = 100 feet (average length of river cell in Wallace Creek)
W = 100 feet (average width of river cell in Wallace Creek)
M = 1 feet (estimated thickness of river sediments)
K = 0.5 ft/d (or 1.8×10^{-4} cm/sec)

This K value is typical of a silty fine sand which is reasonable for the bottom sediment of Wallace Creek near Site 82.

As shown in Figure 5-4, drain cells in the low-lying wetland areas near Wallace Creek were assigned a uniform conductance value (500 ft^2/d). This translates to the following values:

L = 100 feet (average length of drain cell in wetlands along Wallace Creek)
W = 100 feet (average width of drain cell in wetlands along Wallace Creek)
M = 2 feet (estimated thickness of wetland sediments)
K = 0.1 ft/d (or 3.5×10^{-5} cm/sec)

This K value is typical of silts which would be expected in wetlands.

Hydraulic conductivity in layer 1 was uniform at 3 ft/d. This was the average value from the shallow pumping tests at Site 82 (see Appendix A).

Leakance factors (Vcont) in layer 1 had two values: 2×10^{-4} and 3.65×10^{-4} ft/day/ft (see Figure 5-5). Assuming an average water table height of 40 feet, and given a 10 foot confining layer and a thickness of 300 feet in layer 2, the vertical hydraulic conductivity of the confining layer in a quasi 3-D model can be computed from a rearrangement of the following [McDonald & Harbaugh, 1988, pp 5-16):

$$Vcont_{i,j,k+1/2} = (\Delta z_u/2)/K_{zu} + \Delta z_c/K_{zc} + (\Delta z_l/2)/K_{zl}$$

where:

K_{zc} = vertical hydraulic conductivity of the confining unit
Δz_c = thickness of the confining unit (10 feet)
Δz_u = thickness of the upper unit (40 feet)
K_{zu} = vertical hydraulic conductivity of the upper unit (0.3 ft/d)
Δz_l = thickness of the lower unit (300 feet)
K_{zl} = vertical hydraulic conductivity of the lower unit (0.5 ft/d)
$Vcont_{i,j,k+1/2}$ = leakance factor (2×10^{-4} or 3.65×10^{-4} ft/d/ft)

and solving for K_{zc} when $Vcont_{i,j,k+1/2} = 2 \times 10^{-4}$ ft/d/ft:

$$K_{zc} = \Delta z_c / [(1/Vcont_{i,j,k+1/2}) - (\Delta z_u/2)/K_{zu} - (\Delta z_l/2)/K_{zl}]$$

$$K_{zc} = 10 / [(1/2x10^{-4}) - (40/2)/0.3 - (300/2)/0.5]$$

$$K_{zc} = 2.2x10^{-3} \text{ ft/day} = 7.6x10^{-7} \text{ cm/sec}$$

This value is indicative of clay for the confining unit.

Solving for K_{zc} when $Vcont_{i,j,k+1/2} = 3.65x10^{-4}$ ft/d/ft:

$$K_{zc} = 10 / [(1/3.65x10^{-4}) - (40/2)/0.3 - (300/2)/0.5]$$

$$K_{zc} = 4.2x10^{-3} \text{ ft/day} = 1.5x10^{-6} \text{ cm/sec}$$

Where the clay is breached the value is indicative of silts.

5.4.1.2 Pre-Pumping Output

All of the simulated water levels in layer 1 were within 10 feet of their targets at Site 82. Table 5-1 shows that the RM in layer 1 was +1.90 feet, which means that the average simulated head value were 1.9 feet lower than the average of the measured heads in the surficial unit. The ARM was 3.36 feet, the RSD was 3.98 feet, and the RMS was 4.41. As indicated by the value of the RSD for layer 1, about 66% of the simulated heads were within 4 feet (one standard deviation) of their targets and 95% of them were within 8 feet (2 standard deviations).

Figure 5-6 shows the water table contours across Site 82 before any extraction wells were activated. The map shows that Wallace Creek and its surrounding wetlands and tributaries are the main areas of groundwater discharge from the surficial unit near Site 82. The groundwater flow direction from the site generally follows topography due north to Wallace Creek.

5.4.2 Layer 2 - Castle Hayne Aquifer

5.4.2.1 Pre-Pumping Input

The top elevations of layer 2 range from 0 feet (sea level) to -80 feet msl (see Figure 5-6). Bottom elevations in layer 2 range from -30 feet to -140 feet msl (see Figure 5-7). The bottom of layer 2 is sloping from west to east across the study area.

The specified head boundaries in layer 2 are set to about +35 feet msl. The regional gradient is flowing to the northeast (as observed in the BRAGS model).

The uniform hydraulic conductivity value in layer 2 is 5 ft/d. This was the calibrated value and is close to the average from the pumping tests in the Castle Hayne Aquifer (3 ft/d, see Table 3-2).

The locations of the water supply wells near Site 82 are shown on Figure 5-8. The wells were placed in layer 2 and their "steady-state" pumping rates were determined in Section 4.0 (see Table 4-1). Discharge rate values used in MODFLOW are in negative cubic feet per day. No river or drain cells were used in layer 2.

5.4.2.2 Pre-Pumping Output

Figure 5-9 shows the piezometric surface contours in the Castle Hayne around Site 82. The map shows that Wallace Creek and its tributaries are still the main areas of groundwater discharge from the Castle Hayne Aquifer. The effects of nearby water supply wells are also clearly shown on this figure. HP-633 is pumping an average of 109 gpm and HP-709 an average of 85 gpm. It is apparent from this figure that some potential exists for contaminants detected in the Castle Hayne Aquifer at Site 82 to migrate toward supply well HP-633. To date, no contamination has been reported in HP-633.

The contaminants found in the deep wells at Site 82 had apparently been drawn down into the Castle Hayne Aquifer by the former supply well HP-651 which had been taken off-line and subsequently decommissioned due to high concentrations of organic contaminants. The screened interval of HP-651 was from 125 feet to about 200 feet bgs and the contaminants originating at the surface were drawn down into the Castle Hayne Aquifer while HP-651 pumped at a maximum rate of about 270 gpm.

All of the simulated water levels in layer 2 were within 10 feet of their targets with one exception: the simulated water level at water supply well HP-633 was 14 feet below the target. However, the measured target in HP-633 (+15 feet msl) is probably a non-pumping water level. The actual water level near HP-633 when pumping (as simulated in the Site 82) model would be much lower. The simulated value is probably closer to the actual pumping level than this analysis would indicate.

Table 5-1 shows that the RM in layer 2 was 1.48 feet, the ARM was 3.06 feet, the RSD was 3.99 feet, and the RMS was 4.26 feet. As indicated by the value of the RSD for layer 2, about 66% of the simulated heads were within 4 feet (one standard deviation) of their targets and 95% of them were within 8 feet (2 standard deviations). Again, the match for the layer 2 is very similar to that of layer 1.

5.5 Results of Remediation Scenario Simulation

This section describes the inputs and outputs of the remediation simulation. All of the inputs other than extraction wells are identical to the pre-pumping inputs described in section 5.4.

Chlorinated organic compounds have been identified in the surficial and deep groundwater at Site 82. The estimated extents of the plumes within the surficial and deep groundwater evaluated from the July 1997 groundwater are shown in Figures 5-10 and 5-11, respectively. Originally, three shallow and three deep extraction wells were proposed at Site 82 to contain the off-site migration. The pumping rates of the shallow extraction wells were based on data from previous pumping tests conducted within the surficial unit. The pumping rates of the deep extraction wells were estimated based on the pumping rate of the nearby water supply well HP-651. This supply well was decommissioned because of very high chlorinated organic compound concentrations believed to have originated from Sites 6 and 82. Supply well HP-651 had a maximum pumping rate of 270 gpm and was screened between depths of 125 to 200 feet bgs with most of the water being produced from 125 to 155 feet bgs. The well log for HP-651 is shown on Figure 5-12.

With the additional information made available by the Site 82 pumping tests, it became apparent that three shallow wells would not be adequate to contain the off-site migration in the surficial unit. Also, the locations and pumping rates of the three deep extraction wells were revisited. Many intermediate remediation schemes were run with the model where locations and pumping rate of wells were slightly altered before the final remedial scenario was chosen.

5.5.1 Layer 1 -- Surficial Unit

5.5.1.1 Remediation Scenario Input

The soil vapor extraction system (adjacent to the existing extraction well SRW-1) was taken off-line after completion of the soil remediation project. However, groundwater contaminant concentrations in this area remain high. SRW-1 is designed to pump from this "hot spot" area of high VOC concentrations in the groundwater. Five additional shallow (30 feet bgs) extraction wells are proposed to be placed in an east-west line near the northern edge of Site 82. This line of wells is just south of, and adjacent to, the wetland floodplain of Wallace Creek (see Figure 5-13). The easternmost well, SRW-2, is 350 feet due north of SRW-1.

Each of the shallow extraction wells is anticipated to pump up to 5 gpm. The locations of the five additional wells were optimized to contain the shallow groundwater contamination from the area between wells 6MW-32 and 6MW-40. Simulated remedial scenarios with fewer wells were tried unsuccessfully to contain the existing contaminant plumes.

5.5.1.2 Remediation Scenario Output

Figure 5-14 shows the water table contours in a close-up of the Site 82 area with the shallow extraction wells activated. Figure 5-15 shows the capture zones of each well superimposed on the water table contours and the total VOC plume concentrations. The steady-state simulation shows that the proposed wells are able to capture the shallow contamination and prevent further off-site migration toward Wallace Creek.

5.5.2 Layer 2 - Castle Hayne Aquifer

5.5.2.1 Remediation Scenario Input

Three deep (90 to 110 feet bgs) extraction wells were originally proposed for Site 82. As happened with the design for the shallow extraction system, new information became available with the pumping tests at the site (see Appendix A). The main difference between the original and the new designs was that the lower anticipated pumping rates reduced the expected radii of the capture zones. After the pumping test, the pumping rate from SRW-1 was lowered from 150 gpm to about 25 gpm. The other two wells were also projected to have similar rates so the designed screen lengths of SRW-2 and SRW-3 were increased from 20 to 30 feet. This change is expected to increase the expected pumping rate in these two wells to at least 40 gpm.

Also, a modification was made to the original design in which SRW-2 was moved about 400 feet north of its original location (see Figure 5-16). This change brought the two downgradient wells (SRW-2 and SRW-3) into a line perpendicular to groundwater flow which is the typical arrangement for an extraction well system.

5.5.2.2 Remediation Scenario Output

Figure 5-17 shows the steady-state piezometric surface contours in the Castle Hayne around Site 82 with all three deep extraction wells activated. On Figure 5-18, the capture zones around each well show that the contaminants in the Castle Hayne Aquifer can be contained by the proposed remedial design.

5.6 Site 82 Groundwater Flow Model Summary

The Site 82 model describes the three-dimensional pattern of groundwater flow in the surficial unit and Castle Hayne Aquifer (based on the data to which it was calibrated). It achieves the three objectives described in Section 1.1:

- Based on the conceptual model described in Section 2.0, the Site 82 model describes how groundwater flows beneath Site 82 (Objective 1).

- The Site 82 model demonstrates the effects of remedial groundwater withdrawals on the surficial unit and the Castle Hayne Aquifer (Objective 2). The model demonstrates that the relatively low-volume withdrawal rates of the extraction wells will have an extremely localized effect on the water levels in the surficial unit and the Castle Hayne Aquifer.

- The Site 82 model directly addressed the third objective: it clearly showed the relative effectiveness of various site-specific remediation schemes. The locations of the extraction wells in the surficial and in the Castle Hayne Aquifer were finalized by the successful running of the model. "Success" was indicated by complete hydraulic control or "capture" of the contaminant plume. Also, the model indicated that the low volumes of water withdrawn during such remedial actions will not seriously impact the water supply at the Base.

SECTION 5.0 TABLES

TABLE 5-1 - Statistical Summary of
Site 82 Groundwater Flow Model

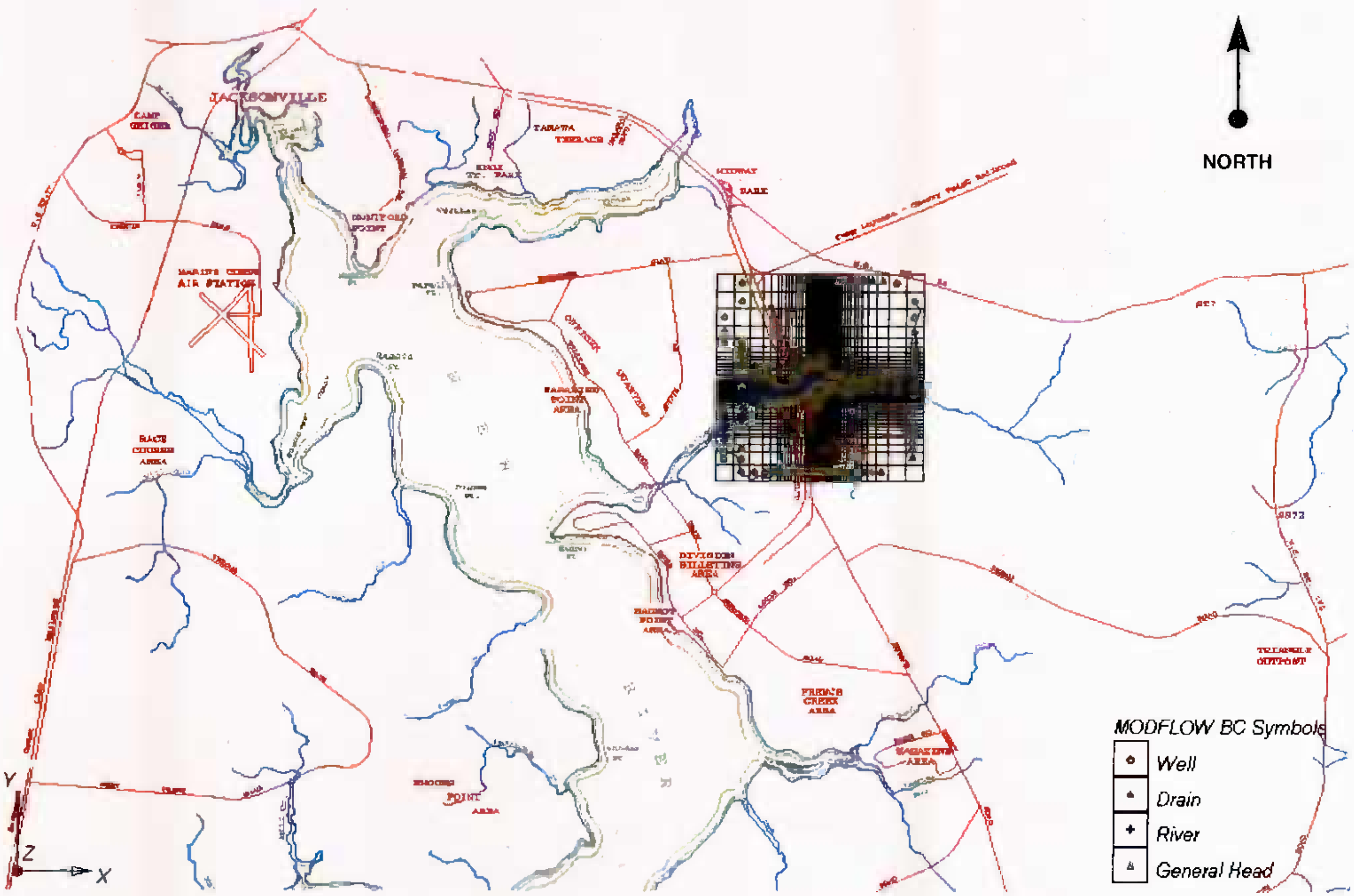
Well Name	Target Head	Model Head	Residual
6-MW6	19.77	19.38	0.39
Site 9-N	12.50	5.06	7.44
Site 74-NE	14.00	20.06	-6.06
Site 74-SE	15.00	18.81	-3.81
HP-612	9.60	4.16	5.44
HP-710	12.00	16.20	-4.20
HP-613/OW-3	6.50	6.02	0.48
Site 9D/OW-4	10.00	10.57	-0.57
HP-654	17.00	11.23	5.77
HP-641	12.00	10.49	1.51
HP-709	13.00	10.80	2.20
X24s2x	5.00	1.17	3.83
HP-648	26.00	18.23	7.77
HP-629	20.00	13.85	6.15
HP-635	6.00	10.10	-4.10
HP-636	16.00	15.55	0.45
HP-653	17.00	13.03	3.97
HP-633	15.00	0.86	14.14
S889/S891	7.00	15.62	-8.62
S889D/S891D	6.00	8.04	-2.04
Site 3D	7.00	10.05	-3.05
Site 3E	26.00	17.30	8.70
Site 3N	22.00	17.56	4.44
6-MW4	21.23	20.22	1.01
6-MW5	20.17	19.65	0.52
6-MW7S	12.63	16.28	-3.65
6-MW8	16.47	16.95	-0.48
6MW-21	17.90	17.48	0.42
6-MW7D	11.56	12.02	-0.46
82MW-3	11.42	10.13	1.29
82MW-1	4.74	0.52	4.22
6GW-33	15.38	9.59	5.79
6GW-34	15.01	8.29	6.72
82MW-2	1.79	0.53	1.26
82MW-30	22.24	12.05	10.19
6GW-1S	17.20	14.47	2.73
6GW-27D	10.13	11.26	-1.13
6GW-28D	10.39	11.53	-1.14
6GW-1D	13.21	13.19	0.02
6GW-2D	16.64	15.15	1.49
6GW-3	16.80	16.49	0.31
6GW-15	18.86	17.22	1.64
6GW-28S	9.73	7.78	1.95
6GW-2S	25.75	20.34	5.41
6GW-26	13.69	12.86	0.83
6GW-11	17.21	16.80	0.41
6GW-23	20.26	19.73	0.53
6GW-25	23.56	21.00	2.56
6GW-31	18.92	17.28	1.64
6GW-16	20.59	19.95	0.64
6GW-32	7.50	3.26	4.24
6GW-30S	7.36	8.92	-1.56
6GW-30D	10.11	12.55	-2.44

6GW-35D	9.11	7.36	1.75
6GW-37D	9.06	10.37	-1.31
6MW-9	25.48	21.43	4.05
6MW-3S	21.86	14.35	7.51
6MW-3D	18.26	15.60	2.66
6GW-15D	14.84	13.45	1.39

```
----- Summary Statistics For Entire Model -----
Residual Mean            = 1.716676
Residual Standard Dev.   = 3.993229
Residual Sum of Squares  = 1114.678404

Absolute Residual Mean   = 3.228911
Minimum Residual         = -8.623124
Maximum Residual         = 14.142834

Observed Range in Head   = 20.910393
Res. Std. Dev./Range     = 0.190969

----- Statistics for Layer   1 -----
Number of Targets        = 33
Residual Mean            = 1.899291
Residual Standard Dev.   = 3.982640
Residual Sum of Squares  = 642.467921

Absolute Residual Mean   = 3.364606
Minimum Residual         = -8.623124
Maximum Residual         = 10.191839

Observed Range in Head   = 20.910393
Res. Std. Dev./Range     = 0.190462

----- Statistics for Layer   2 -----
Number of Targets        = 26
Residual Mean            = 1.484895
Residual Standard Dev.   = 3.994625
Residual Sum of Squares  = 472.210483

Absolute Residual Mean   = 3.056682
Minimum Residual         = -4.202745
Maximum Residual         = 14.142834

Observed Range in Head   = 17.369942
Res. Std. Dev./Range     = 0.229973
```

SECTION 5.0 FIGURES

Figure 5-1 - Finite Difference Grid Location Map
Site 82 Groundwater Flow Model
CTO-0140 - MCB, Camp Lejeune, North Carolina



02013AAB3Y

Figure 5-2 Finite Difference Grid
Site 82 Groundwater Flow Model
CTO-0140 - MCB, Camp Lejeune, North Carolina

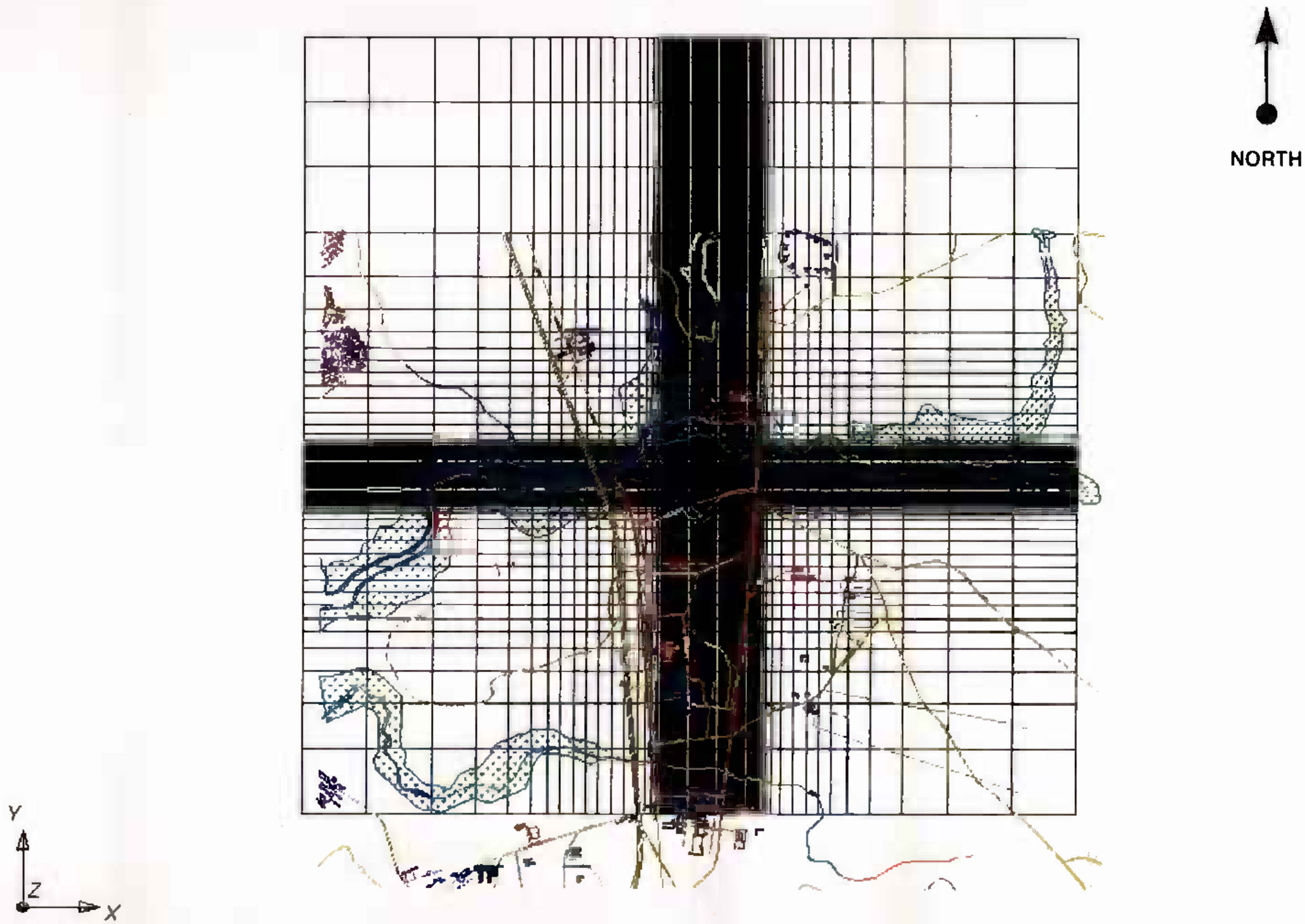


Figure 5-3 Bottom Elevation of Layer 1 (Surficial Unit)
Site 82 Groundwater Flow Model
CTO-0140 - MCB, Camp Lejeune, North Carolina

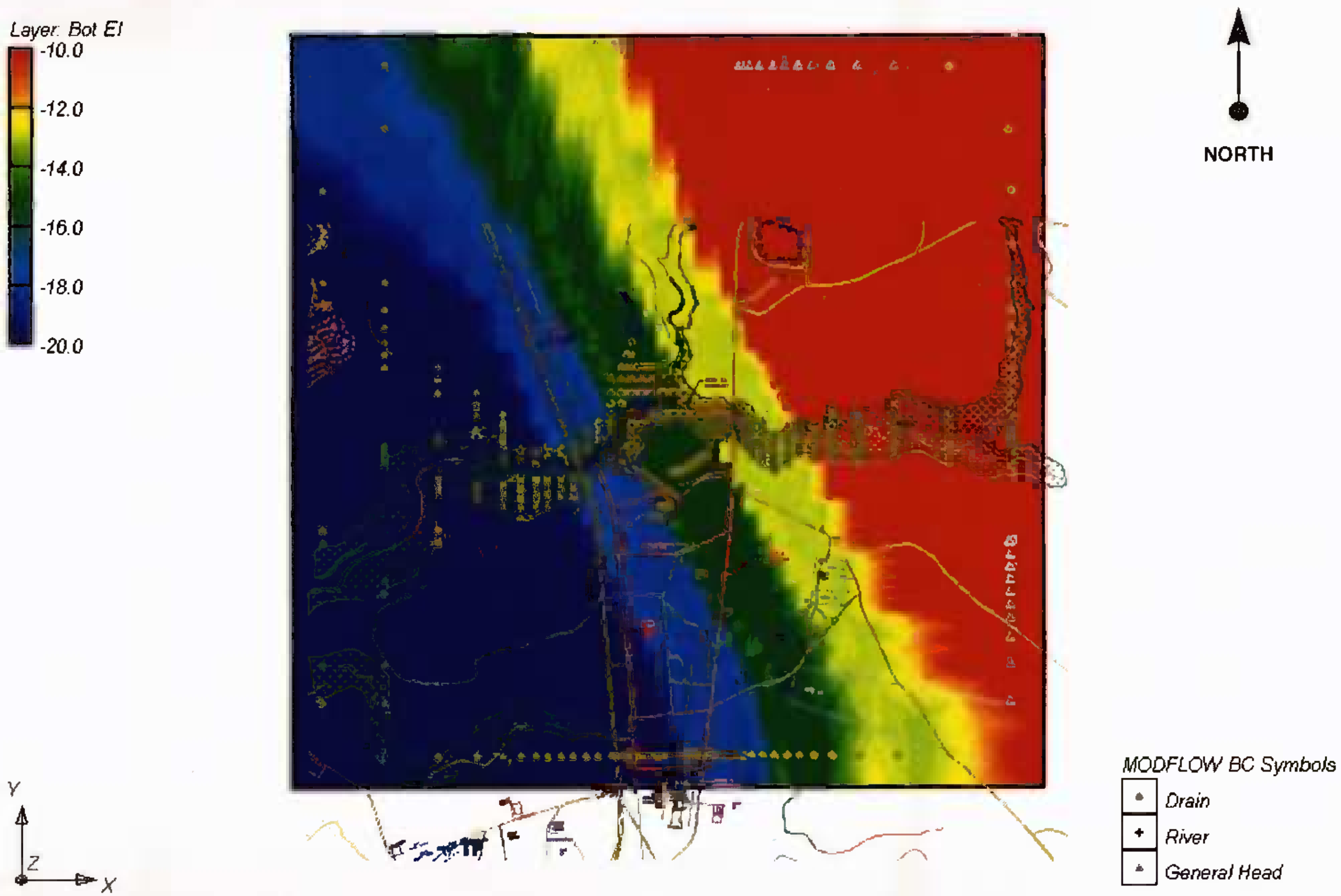


Figure 5-4 General Head Boundary, River, and Drain Cells in Layer 1
Site 82 Groundwater Flow Model
CTO-0140 - MCB, Camp Lejeune, North Carolina

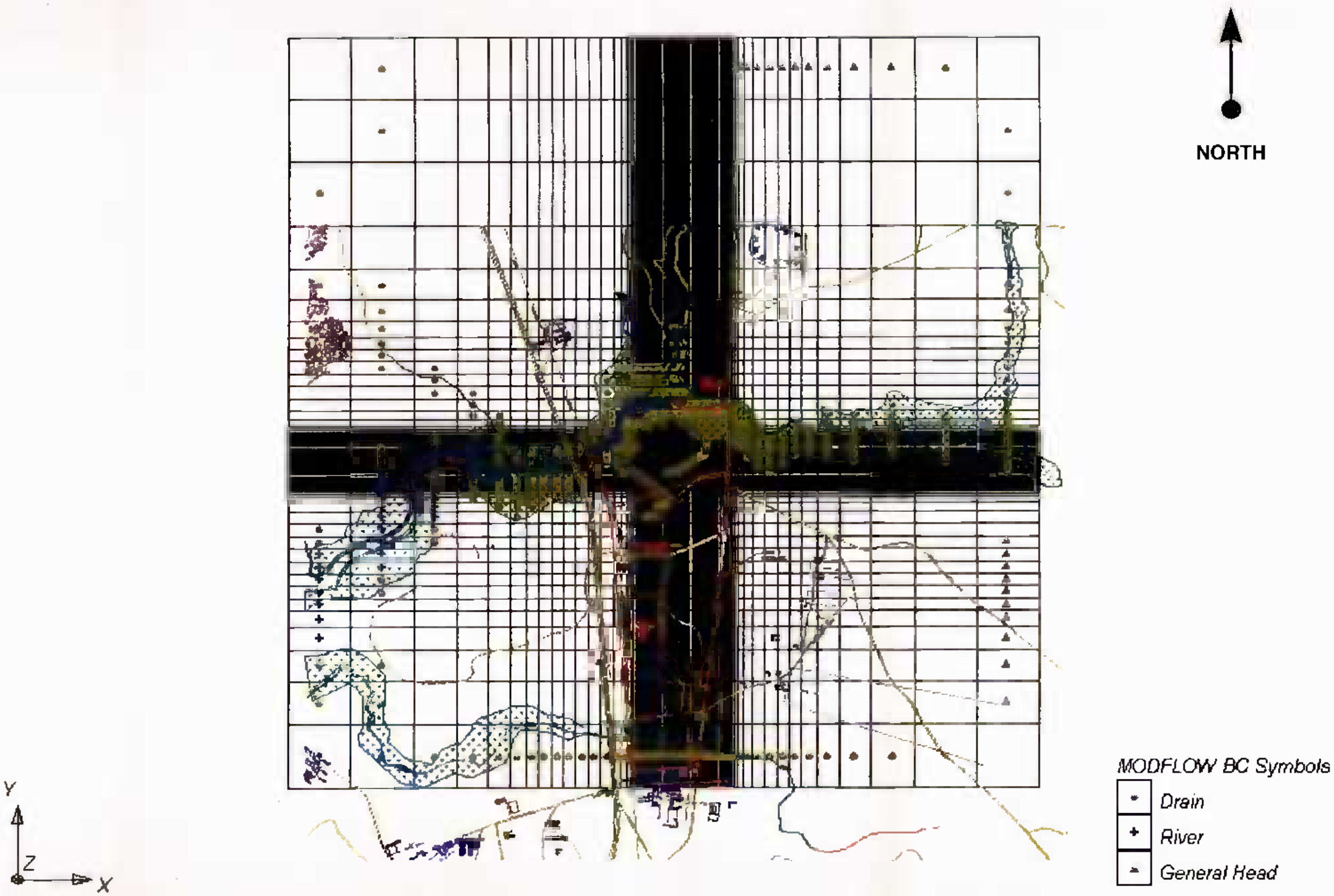


Figure 5-5 Leakance Factor in Layer 1
Site 82 Groundwater Flow Model
CTO-0140 - MCB, Camp Lejeune, North Carolina

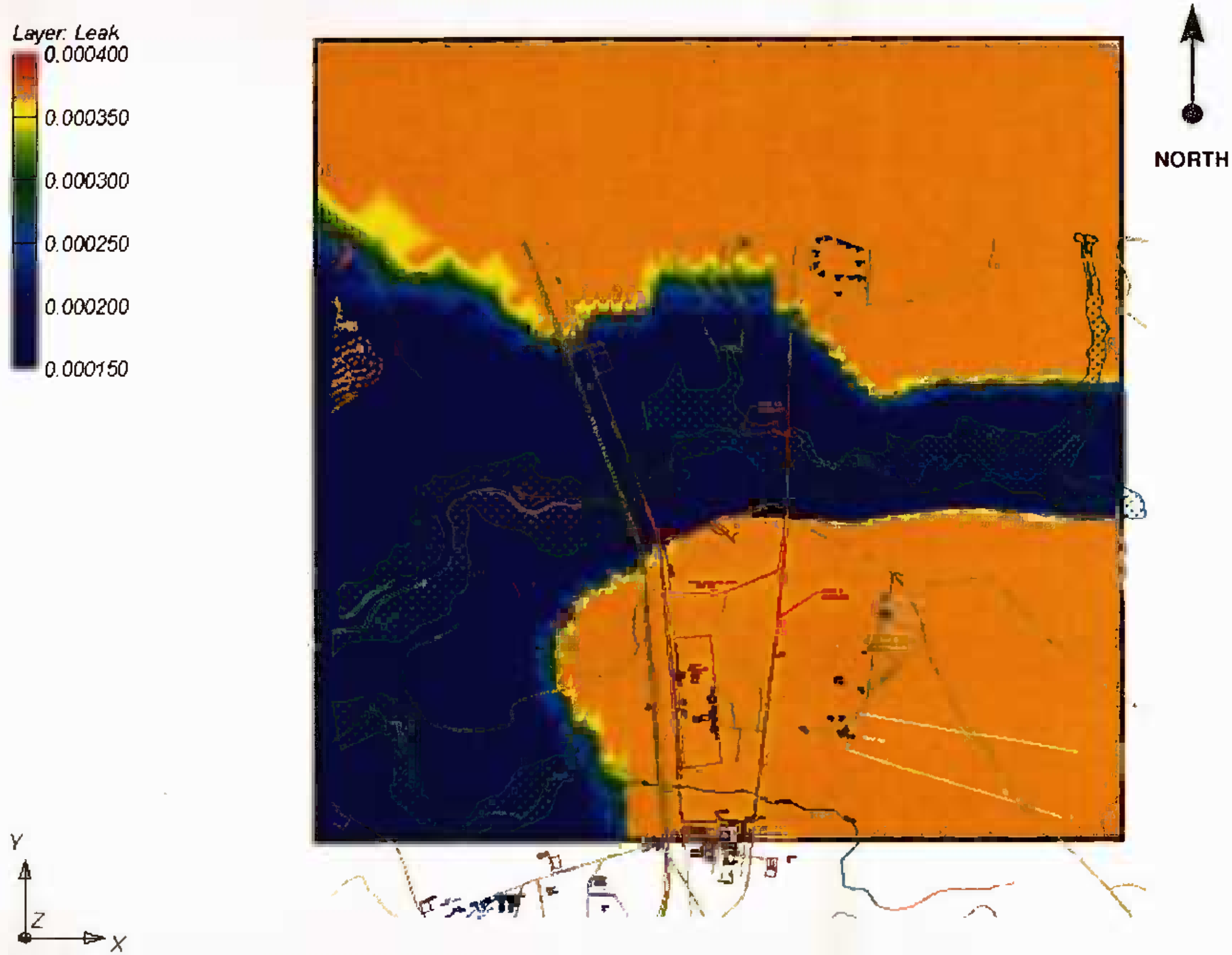


FIGURE 5-6 Simulated Water Table Surface Contours (Pre-Remediation) in Layer 1
Site 82 Groundwater Flow Model
CTO-0140 - MCB, Camp Lejeune, North Carolina

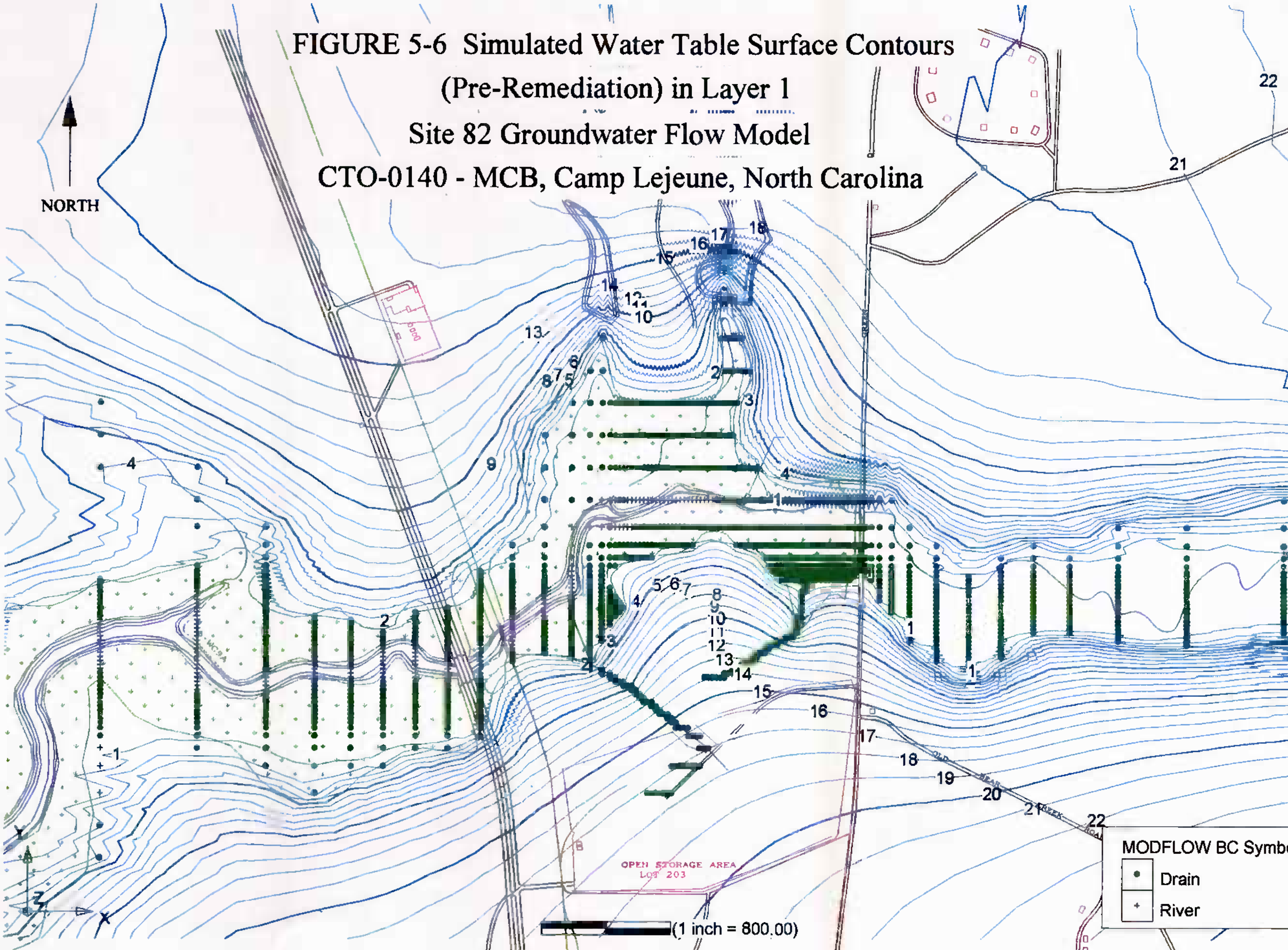


Figure 5-7 Bottom Elevation of Layer 2 (Castle Hayne Aquifer)
Site 82 Groundwater Flow Model
CTO-0140 - MCB, Camp Lejeune, North Carolina

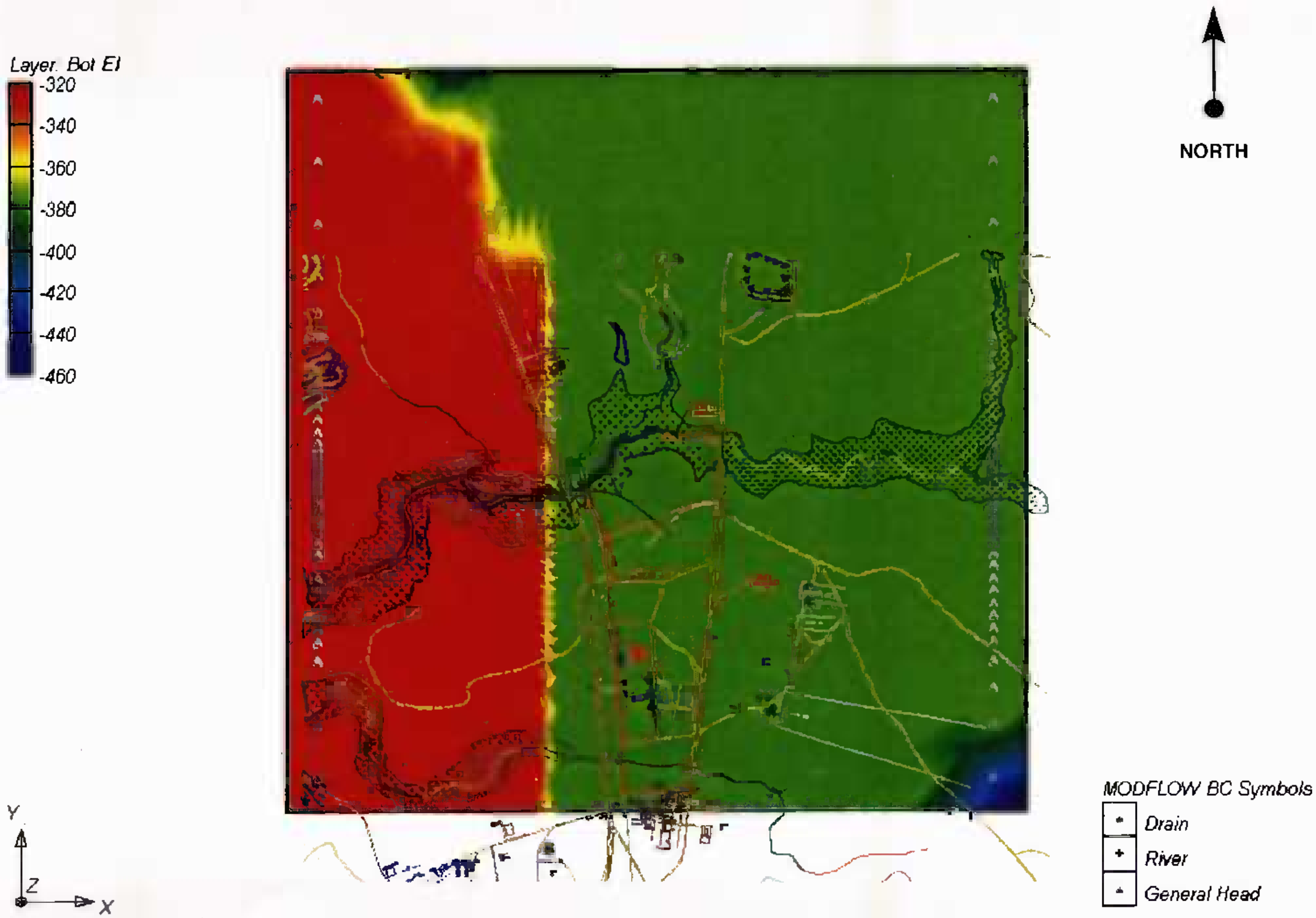


Figure 5-8 Water Supply Well Cells in Layer 2 (Castle Hayne Aquifer)
Site 82 Groundwater Flow Model
CTO-0140 - MCB, Camp Lejeune, North Carolina

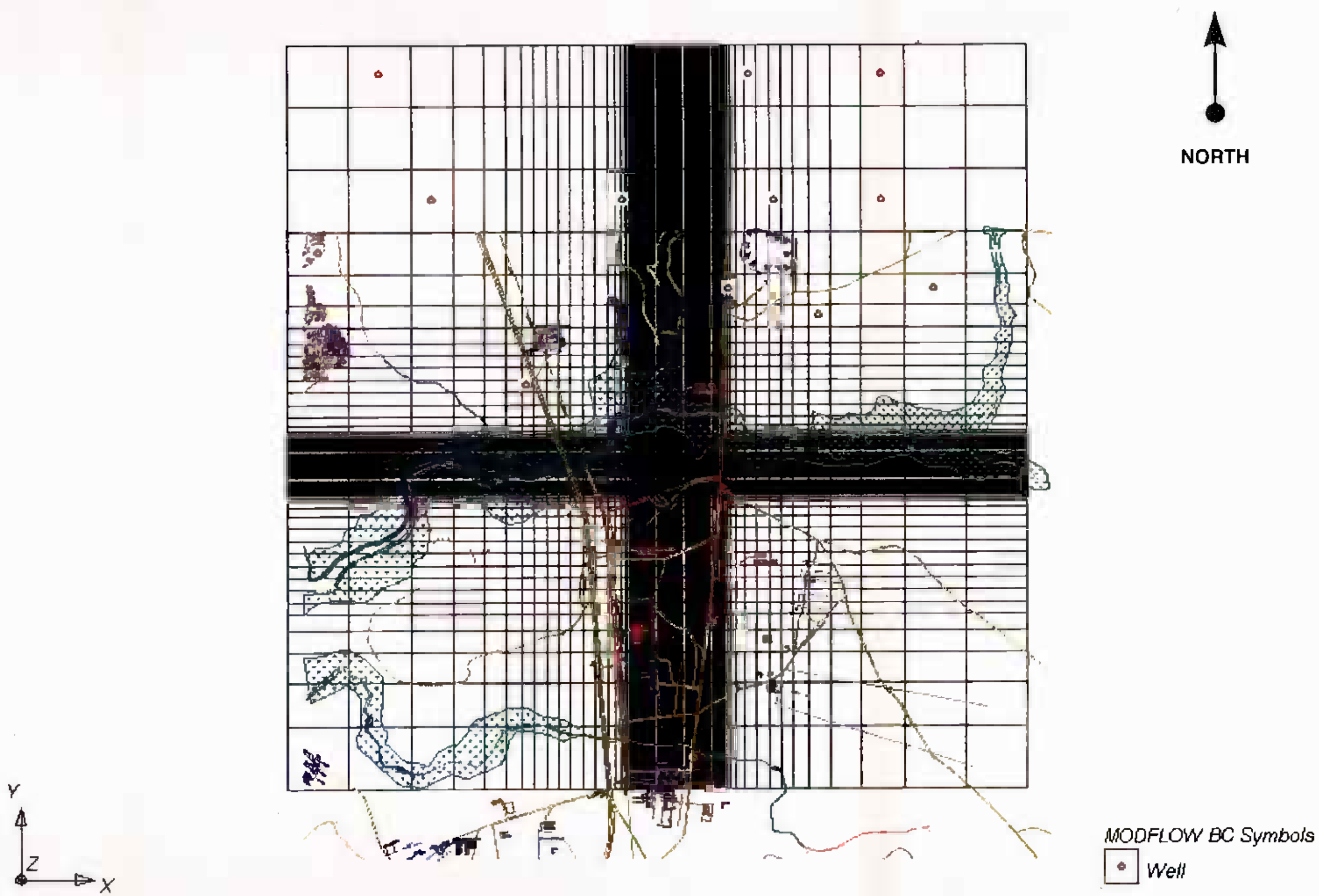
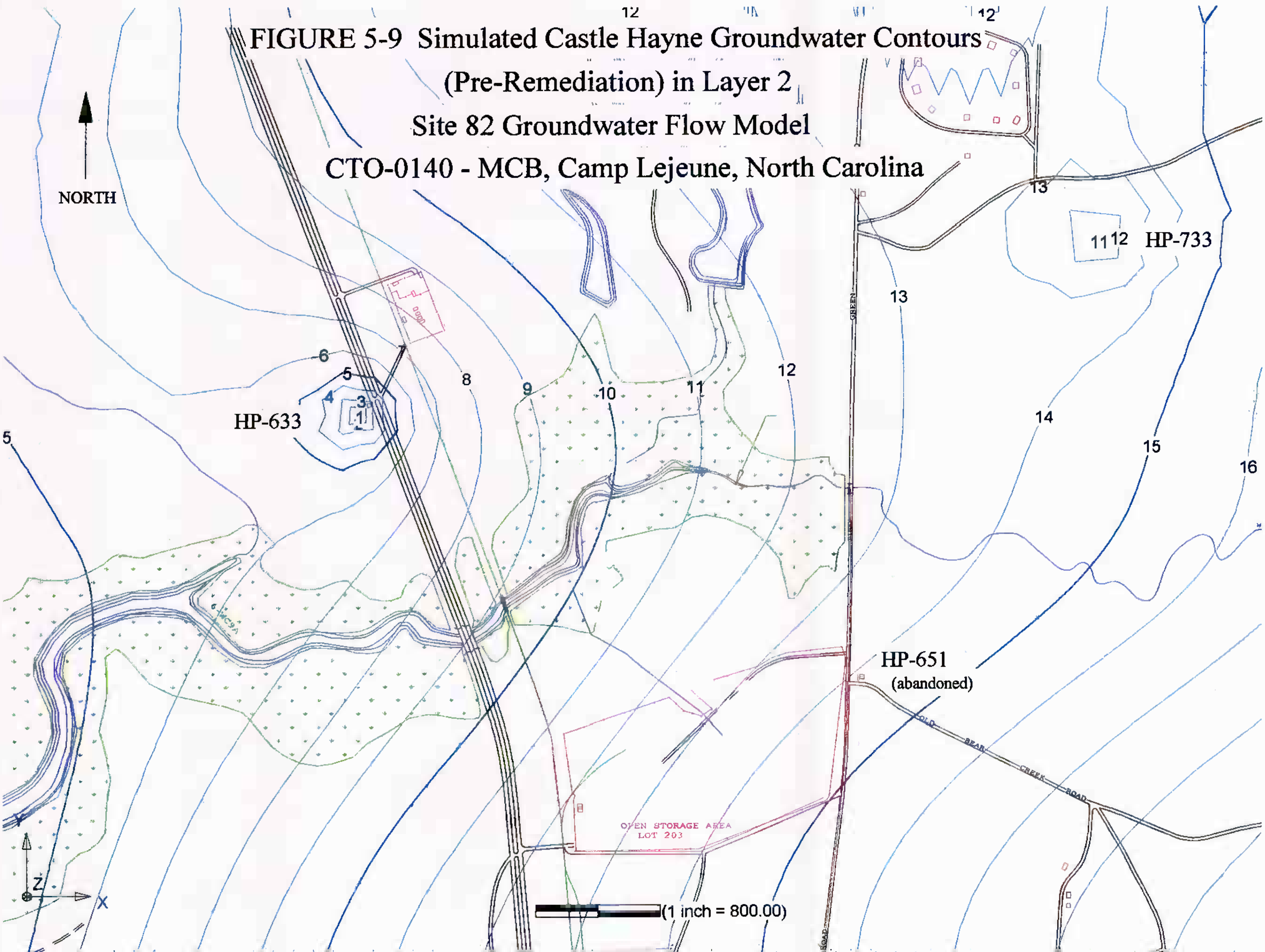


FIGURE 5-9 Simulated Castle Hayne Groundwater Contours (Pre-Remediation) in Layer 2
Site 82 Groundwater Flow Model
CTO-0140 - MCB, Camp Lejeune, North Carolina



LEGEND

GW08 ⊕ 4426.00 — Shallow Monitoring Well Location with Total VOC Concentration

Estimated Extent of Contaminant Concentrations

NOTES:

1. Concentrations presented in micrograms per liter

Scale: 500 0 500 1000 Feet

N E S W

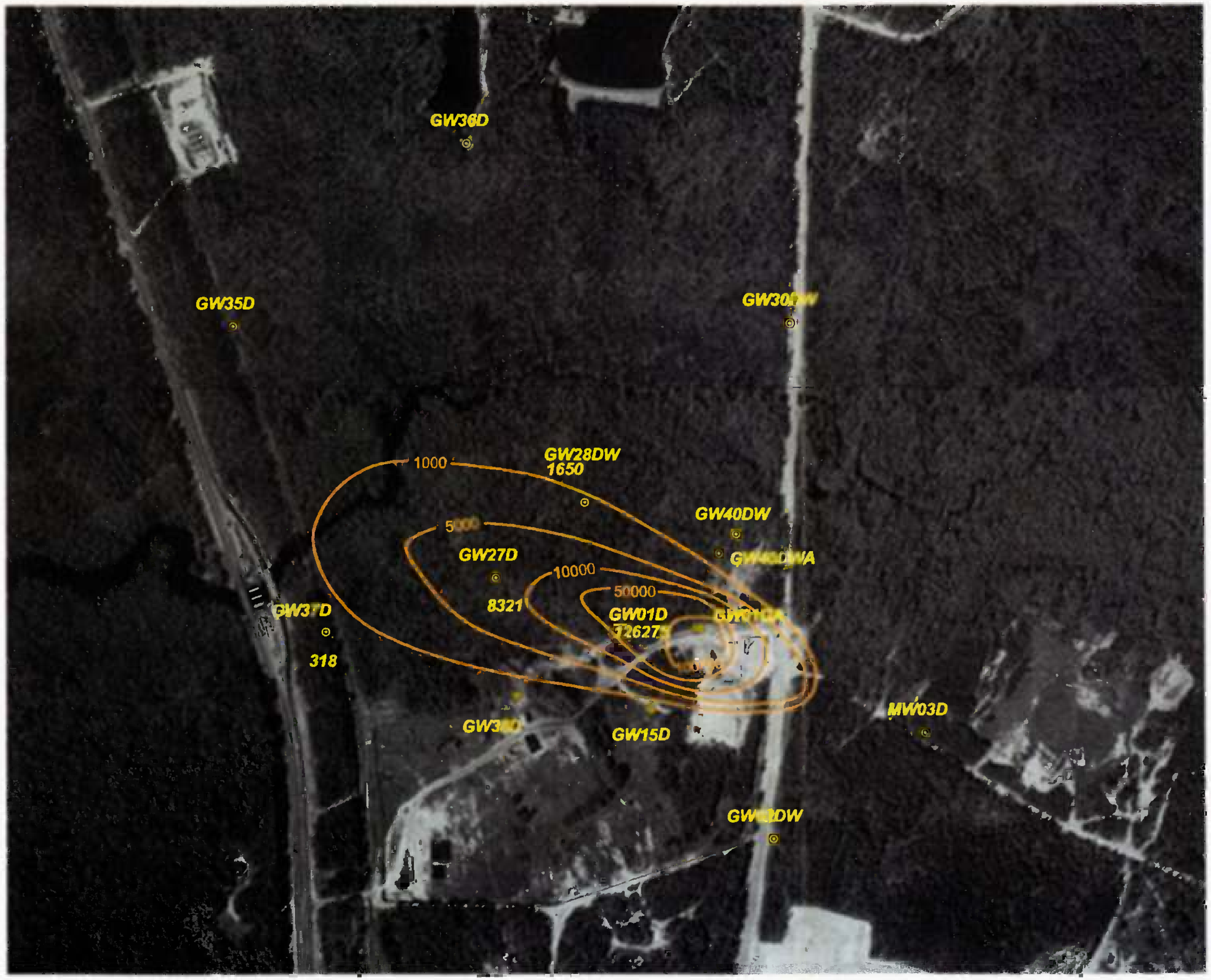
Loc ID	Parameter	Result	Units	Date
IR06-GW34	TRICHLOROETHENE	310	UG/L	07/24/97
	TETRACHLOROETHENE	170	UG/L	07/24/97
	1,1,2,2-TETRACHLOROETHANE	5600	UG/L	07/24/97
IR06-GW28	TRICHLOROETHENE	22	UG/L	07/25/97
	TETRACHLOROETHENE	7	UG/L	07/25/97
IR06-GW16	CHLOROBENZENE	2700	UG/L	07/27/97
	1,1,2,2-TETRACHLOROETHANE	11	UG/L	07/27/97
IR06-GW32	VINYL CHLORIDE	16	UG/L	07/27/97
	TETRACHLOROETHENE	110	UG/L	07/27/97
	1,2-DICHLOROETHENE (TOTAL)	1500	UG/L	07/27/97
	TRICHLOROETHENE	2800	UG/L	07/27/97

MARINE CORPS BASE, CAMP LEJEUNE NORTH CAROLINA

Volatile Organic Compounds in Shallow Groundwater
Operable Unit No. 2 - Site 82
Groundwater Flow Model
CTO-0140

Baker

FIGURE 5-10



LEGEND

GW28DW Deep Monitoring Well Location with Total VOC Concentration 1650

Estimated Extent of Contaminant Concentrations

NOTES:

1. Concentrations presented in micrograms per liter

Scale

300 0 300 600 900 1200 Feet

N W E S

Loc ID	Parameter	Result	Units	Date
IR06-GW27DW	1,1-DICHLOROETHENE	11	UG/L	07/22/97
	VINYL CHLORIDE	110	UG/L	07/22/97
	TRICHLOROETHENE	3400	UG/L	07/22/97
	1,2-DICHLOROETHENE (TOTAL)	4800	UG/L	07/22/97
IR06-GW37D	TRICHLOROETHENE	88	UG/L	07/23/97
	1,2-DICHLOROETHENE (TOTAL)	230	UG/L	07/23/97
IR06-GW28DW	1,2-DICHLOROETHENE (TOTAL)	550	UG/L	07/25/97
	TRICHLOROETHENE	1100	UG/L	07/25/97
IR06-GW01D	METHYLENE CHLORIDE	8	UG/L	07/26/97
	VINYL CHLORIDE	320	UG/L	07/26/97
	1,1-DICHLOROETHENE	57	UG/L	07/26/97
	TETRACHLOROETHENE	890	UG/L	07/26/97
	1,2-DICHLOROETHENE (TOTAL)	28000	UG/L	07/26/97
	TRICHLOROETHENE	97000	UG/L	07/26/97

**MARINE CORPS BASE, CAMP LEJEUNE
NORTH CAROLINA**

Volatile Organic Compounds in Deep Groundwater
Operable Unit No. 2 - Site 82
Groundwater Flow Model
CTO-0140

Baker

FIGURE 5-11

FIGURE 5-12 -- Well Log for Supply Well HP-651 (a.k.a. Well #9)
(Reproduced from the best available copy.)

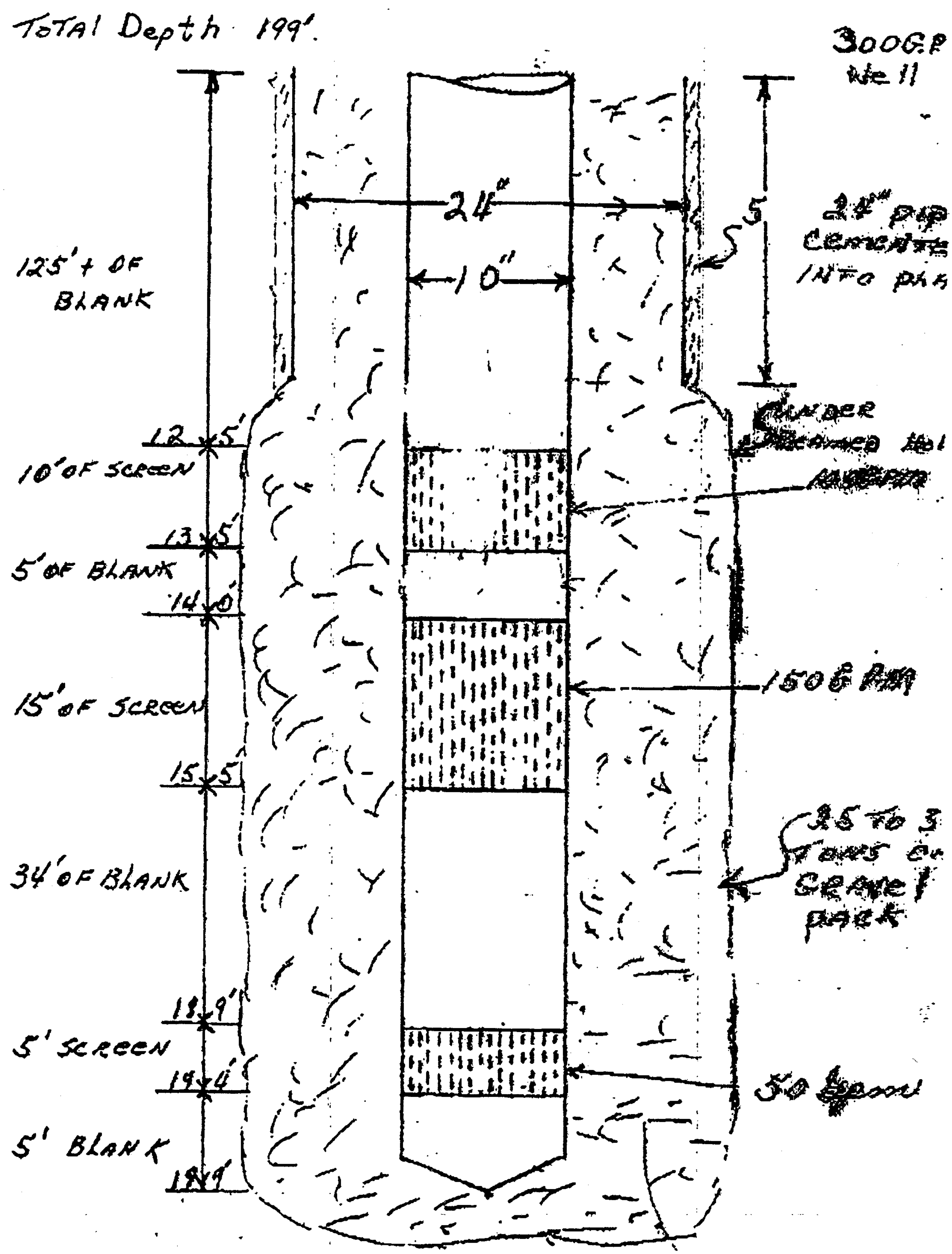


FIGURE 5-13 Shallow Extraction Wells in the Surficial Unit (Layer 1)

Site 82 Groundwater Flow Model

CTO-0140 - MCB, Camp Lejeune, North Carolina

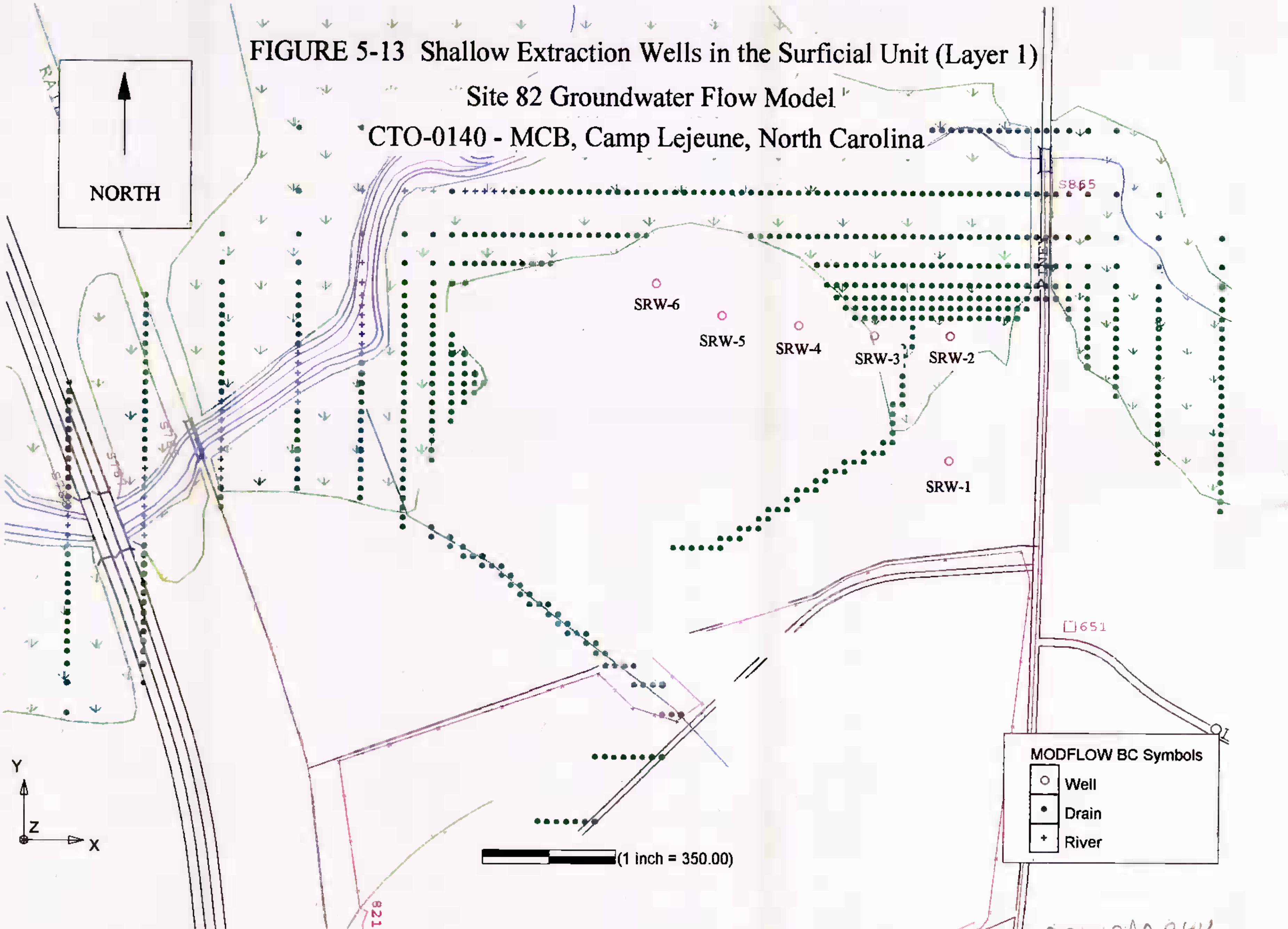
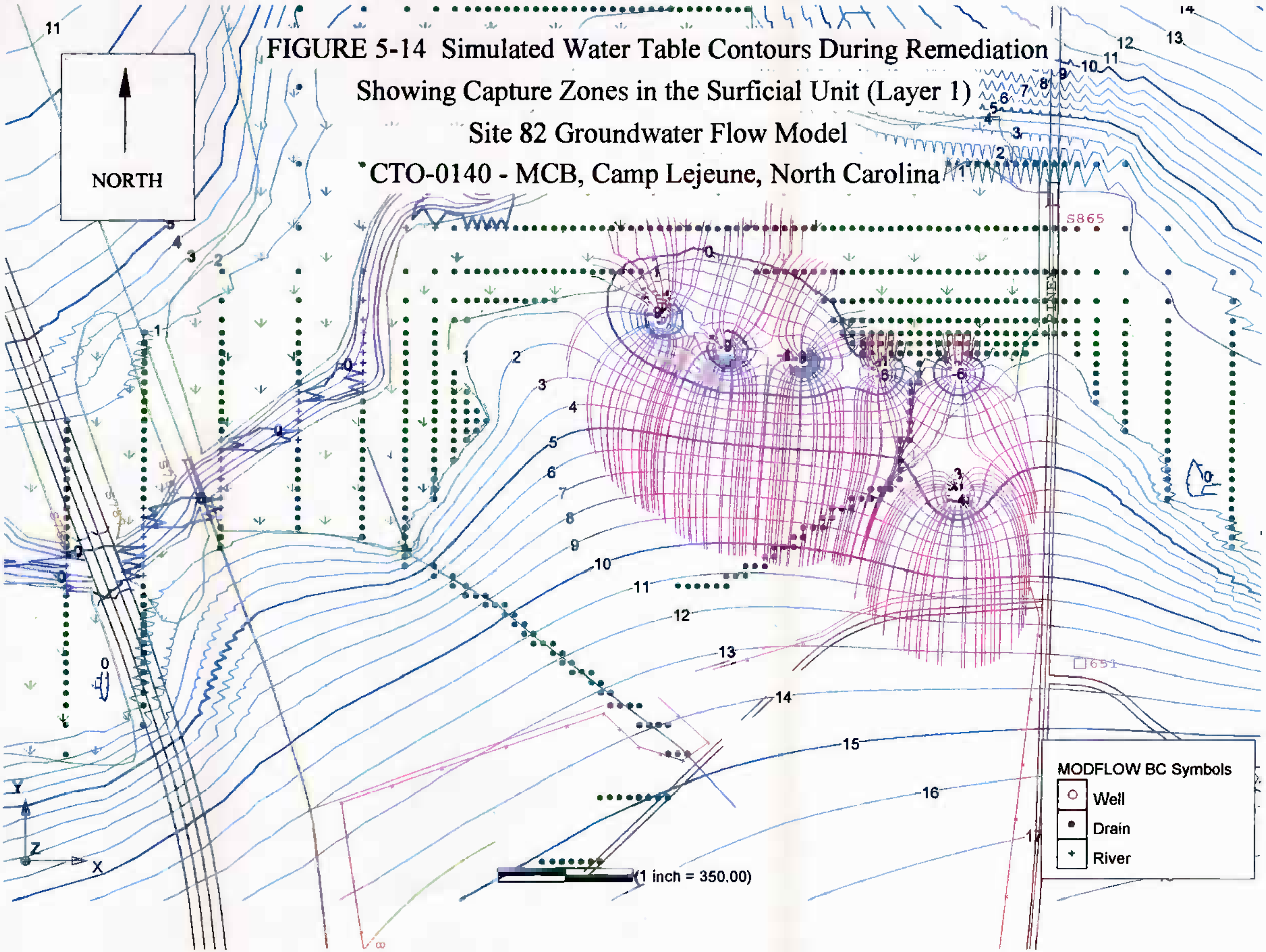
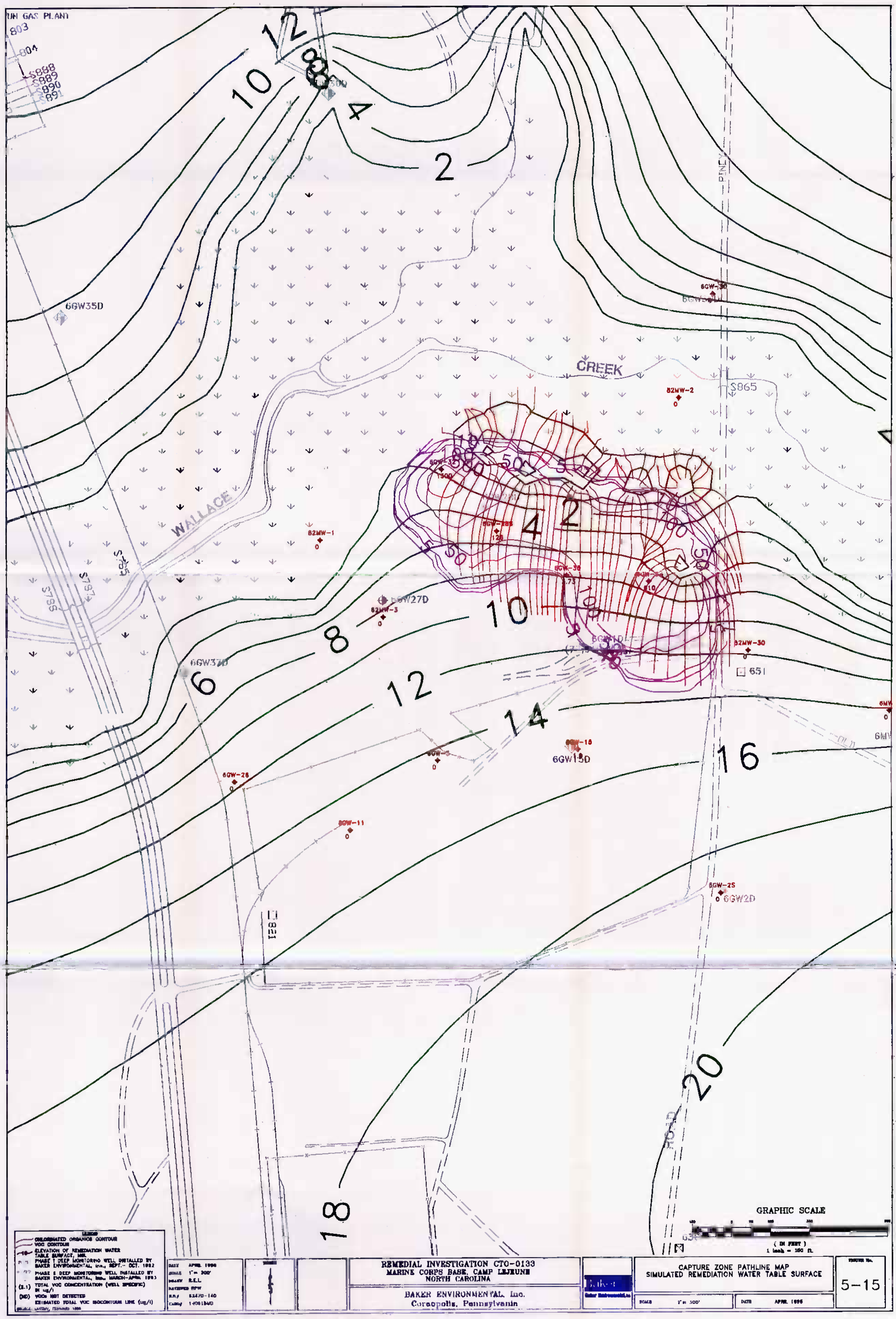


FIGURE 5-14 Simulated Water Table Contours During Remediation
Showing Capture Zones in the Surficial Unit (Layer 1)
Site 82 Groundwater Flow Model
CTO-0140 - MCB, Camp Lejeune, North Carolina





GRAPHIC SCALE
(IN FEET)
1 inch = 150 ft.
LEGEND
CHLORINATED ORGANICS CONTOUR
VOC CONTOUR
ELEVATION OF REMEDIATION WATER TABLE SURFACE, MSL
PHASE I DEEP MONITORING WELL INSTALLED BY BAKER ENVIRONMENTAL, Inc., SEPT.- OCT. 1992
PHASE II DEEP MONITORING WELL INSTALLED BY BAKER ENVIRONMENTAL, Inc., MARCH-APRIL 1993
(2.1) TOTAL VOC CONCENTRATION (WELL SPECIFIC) IN ug/l
(ND) VOCs NOT DETECTED
ESTIMATED TOTAL VOC ISOCONTOUR LINE (ug/l)
DATE APRIL 1996
SCALE 1" = 300'
R.E.L.
RPW
62470-140
1406184D
REMEDIAL INVESTIGATION CTO-0133
MARINE CORPS BASE, CAMP LEJEUNE
NORTH CAROLINA
BAKER ENVIRONMENTAL, Inc.
Coraopolis, Pennsylvania
CAPTURE ZONE PATHLINE MAP
SIMULATED REMEDIATION WATER TABLE SURFACE
SCALE 1" = 300'
DATE APRIL 1996
5-15
WALLACE
CREEK
PINEY
6GW35D
6GW37D
6GW27D
6GW15D
6GW2D
821
651
S865

FIGURE 5-16 Deep Extraction Wells in the Castle Hayne Aquifer (Layer 2)
Site 82 Groundwater Flow Model
CTO-0140 - MCB, Camp Lejeune, North Carolina

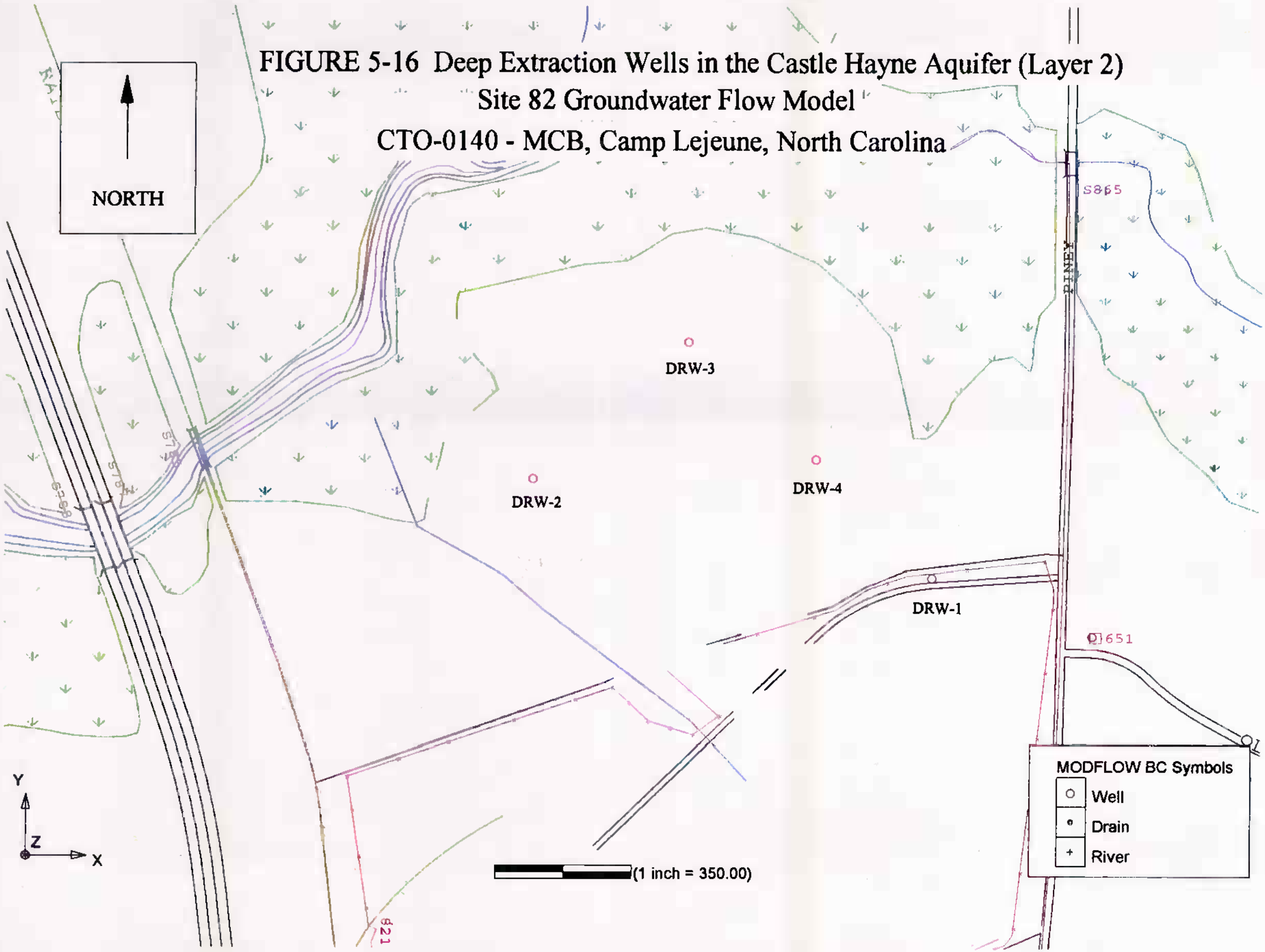
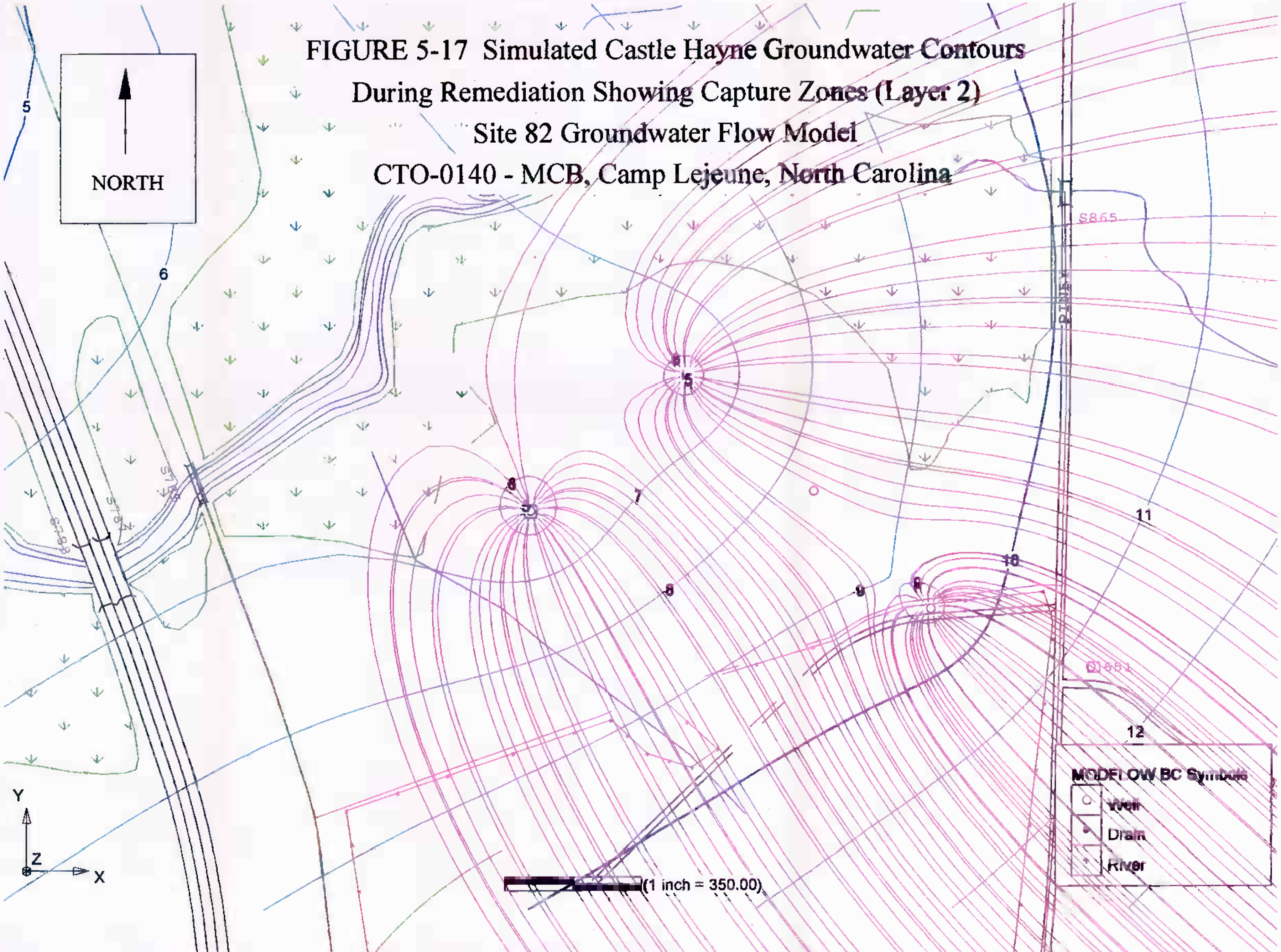
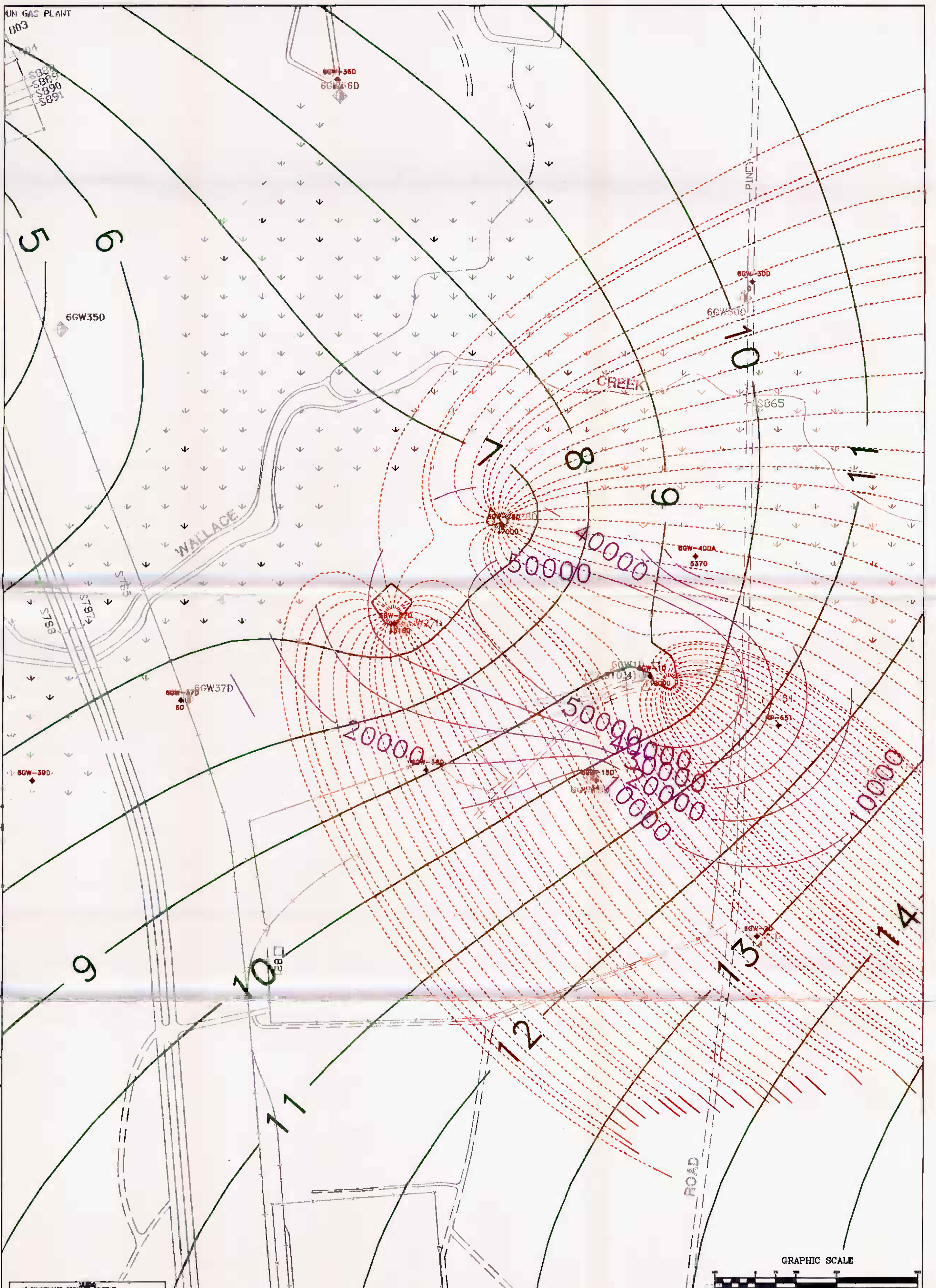


FIGURE 5-17 Simulated Castle Hayne Groundwater Contours During Remediation Showing Capture Zones (Layer 2)
Site 82 Groundwater Flow Model
CTO-0140 - MCB, Camp Lejeune, North Carolina





CHLORINATED ORGANICS CONTOUR
VOC CONTOUR
ELEVATION OF REMEDIATION WATER TABLE SURFACE, MSL
PHASE I DEEP MONITORING WELL INSTALLED BY BAKER ENVIRONMENTAL, Inc., SEPT.- OCT. 1992
PHASE II DEEP MONITORING WELL INSTALLED BY BAKER ENVIRONMENTAL, Inc., MARCH-APRIL 1993
(2.1) TOTAL VOC CONCENTRATION (WELL SPECIFIC) IN ug/l
(ND) VOCs NOT DETECTED
ESTIMATED TOTAL VOC ISOCONTOUR LINE (ug/l)

DATE APRIL 1996
SCALE 1" = 300'
DRAWN R.E.L.
B.E.J. 19153-22-BRN
CADD# 140518MD

BRAGS, CTO-0140
MARINE CORPS BASE, CAMP LEJEUNE
NORTH CAROLINA

BAKER ENVIRONMENTAL, Inc
Coraopolis, Pennsylvania

Baker

CAPTURE ZONE PATHLNE MAP
SIMULATED REMEDIATION CASTLE
HAYNE PIEZOMETRIC SURFACE

SCALE 1" = 300'
DATE APRIL 1996

FIGURE No.
5-18

6.0 CONCLUSIONS AND RECOMMENDATIONS

The modeling effort described herein was successful in achieving the three objectives stated at the outset of this report. The objectives were to:

- Describe how groundwater flows beneath the entire Base as well as under individual sites of concern.
- Demonstrate the effects of groundwater withdrawals (supply and remedial) on the aquifers in question (most notably the surficial unit and the Castle Hayne Aquifer).
- Predict the relative effectiveness of various remediation schemes at individual sites (including Site 82).

The two groundwater flow models were intended to be "working" models, that is, they were meant to be transferred into the hands of Base personnel (or their representatives) to update and modify to site-level work or as new information becomes available. The updated models will be effective decision-making tools for optimal groundwater resource management, protection, and restoration. The models can be used to determine the relative effectiveness of various remedial scenarios at individual sites around the Base.

The BRAGS groundwater flow model presented herein portrays the three-dimensional pattern of groundwater flow within the surficial units and the Castle Hayne Aquifer (based on the data to which it was calibrated). The model reasonably predicts the elevation and flow direction of the surficial and Castle Hayne groundwater in many areas around the Base where no data currently exist. The BRAGS model also demonstrates that discharge to the New River is the controlling factor on flow directions in the Castle Hayne Aquifer in the vicinity of Camp Lejeune. The model output indicates that the relatively high-volume withdrawal rates of the supply wells have a localized effect on the water levels in the Castle Hayne.

One of the concerns that initiated this modeling effort was that the potential number of pump and treat remedial actions at the Base may negatively impact the supply of available groundwater. The BRAGS model strongly indicated that the low volumes of water withdrawn from the surficial unit and/or the Castle Hayne Aquifer during such remedial actions will not noticeably affect the groundwater supply at the Base; however, large numbers of actively pumping water supply wells in small areas have the potential to induce saltwater intrusion into the upper Castle Hayne Aquifer. This effect is most pronounced in Paradise Point along Brewster Boulevard. Actively pumping water supply wells should not be grouped together in small areas but should be spread out in a line perpendicular to the ambient flow direction (not parallel to it) to avoid this situation.

The Site 82 model describes the three-dimensional pattern of groundwater flow in the surficial unit and Castle Hayne Aquifer. The Site 82 model demonstrates the effects of proposed remedial groundwater withdrawals on the surficial unit and the Castle Hayne Aquifer. The model also demonstrates that the relatively low-volume withdrawal rates of the extraction wells will have an extremely localized effect on the water levels in the surficial unit and the Castle Hayne Aquifer.

The Site 82 model directly addressed the third objective: it clearly showed the relative effectiveness of various site-specific remediation schemes. The locations of the extraction wells in the surficial and in the Castle Hayne Aquifer were finalized by the successful running of the model. "Success" was indicated by complete hydraulic control or "capture" of the contaminant plume. Also, the model indicated that the low volumes of water withdrawn during such remedial actions will not noticeably affect the groundwater supply at the Base.

The groundwater flow models described herein will be useful in managing the future RI activities at the Base. The BRAGS model will be especially useful for determining the groundwater flow patterns in areas where no data currently exists and it gives a regional perspective on site-specific modeling. Future groundwater flow and/or contaminant transport modeling done at the site level should be coordinated with the BRAGS groundwater flow model so that the "big picture" of the groundwater flow is consistent across the Base.

It is strongly recommended that the additional hydrogeologic and chemical data collected from the on-going remediation activities and long-term monitoring at Site 82 be incorporated into the Site 82 groundwater flow model. At that time, the Site 82 groundwater flow model should be converted to fully three-dimensional so that it is capable of modeling contaminant transport. From future modeling efforts (which should include actual pumping rates, updated groundwater elevations, and contaminant concentrations) recommendations can be provided to address the question of complete capture and the necessity of additional wells at Site 82.

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APPENDIX A
SITE 82 PUMPING TEST DATA EVALUATION

AQUIFER TESTING REPORT

Site 82 -- Piney Green VOC Area

Marine Corps Base, Camp Lejeune, North Carolina

Baker Environmental, Inc.

July, 1996

AQUIFER TESTING REPORT

Site 82 -- Piney Green VOC Area

Marine Corps Base, Camp Lejeune, North Carolina

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LIST OF ATTACHMENTS

1.0 INTRODUCTION AND OBJECTIVES

This report describes two aquifer tests performed as "pilot" tests in two extraction (recovery) wells at Site 82. One recovery well was screened within the surficial unit (SRW-1) near a "hot spot" of volatile organic compound (VOC) contamination and the other near a similar VOC "hot spot" within the Castle Hayne aquifer (DRW-1). These tests were performed so that an accurate estimate of discharge volume of contaminated groundwater might be provided to the operator of the treatment plant (OHM Corp.) and so that the aquifer characteristics of both hydrologic units (surficial and Castle Hayne) could be quantified. The data from the tests were evaluated and the results used to update the Site 82 groundwater flow model.

OHM Corp. (OHM) installed the two recovery wells and the observation piezometers. Well completion logs for the wells and piezometers were not generated by OHM. OHM authored the pumping and recovery test procedures including the methodology (see Attachment A).

2.0 RECOVERY WELL AND OBSERVATION PIEZOMETER INSTALLATION

Baker Environmental, Inc. (Baker) authored the conceptual design of the pump and treat system at the Site 82 -- the Piney Green VOC Area (Baker, 1993) and identified tentative locations of several recovery wells. These wells were to be located within the shallow and deep "hot spots" of VOC contamination identified by several rounds of groundwater sampling and analysis. OHM installed two recovery wells, one in the surficial unit and one in the upper Castle Hayne aquifer in the areas of highest VOC contamination. OHM also installed several shallow and deep observation piezometers. No well completion logs were generated by OHM for the recovery wells nor for the observation piezometers; only the driller's logs (required by the State of North Carolina) were filed with the state.

2.1 Shallow Well and Piezometers

The shallow recovery well (SRW-1) and three shallow observation piezometers (SP-1, SP-2, and SP-3) were installed near an existing shallow monitoring well 6GW-34 (see Figure 2-1). The installation of SRW-1 was performed by OHM during February and March 1995 with an OHM geologist providing logging and well installation inspection services. The piezometers were also installed by OHM in December 1995. Completion logs for SRW-1 and the shallow piezometers were not generated by OHM.

SRW-1 is a 6" diameter stainless steel well screened (0.010" slots) from a depth of 15 feet to 35 feet below ground surface (bgs). The location of SRW-1 was chosen so that existing wells (particularly 6GW-34 and 6GW-1S) could be used during the test as observation wells and so that it could later be used as a recovery well at the shallow "hot spot" of high VOC concentrations near the recently completed soil vapor extraction (SVE) system. The shallow observation piezometers (SP-1, SP-2, and SP-3) consist of 2" PVC and were screened (slot size unknown) from 15 to 35 feet bgs as was SRW-1.

The purpose of the piezometers was to provide monitoring points close enough to the pumping well to observe the water level changes associated with the two phases (drawdown and recovery) of the shallow pumping test.

2.2 Deep Well and Piezometers

The deep recovery well (DRW-1) and two deep observation piezometers (DP-1 and DP-2) were installed near existing deep monitoring wells 6GW-1D, 6GW-1DA and 6GW-1DB (see Figure 2-2). The installation was performed during March 1995 with an OHM geologist providing logging and well installation inspection services. The deep piezometers were installed in December 1995 by OHM. Completion logs were not generated by OHM for DRW-1 and the deep piezometers.

DRW-1 was originally designed to be screened from 90 to 110 feet bgs. However, heaving sands were encountered at the original install depth and the well was not installed as originally designed. After three tries to install the well to a total depth of 110 feet bgs, sand caved in around the screen up to 92 feet bgs (18 feet of caved-in sand). OHM decided to pull the casing up 10 feet so that the screen would be from about 81 to 101 feet bgs. However, this still left at least 9 feet of screen surrounded by caved-in sand (from 101 to 92 feet bgs).

DRW-1 is a 6" diameter stainless steel well screened (0.010" slots) from a depth of 81 feet to 101 feet below ground surface (bgs). The location of DRW-1 was chosen so that existing wells (particularly 6GW-1D and 6GW-1S) could be used during the test as observation wells and so that it could later be used as a recovery well at the "hot spot" of high VOC concentrations at depth (90' to 110') in the Castle Hayne aquifer. The deep observation piezometers (DP-1 and DP-2) consist of 2" PVC and were screened from 80 to 100 feet bgs. The purpose of the observation wells was to provide monitoring points close enough to the pumping well to observe the water level changes associated with the two phases (drawdown and recovery) of the aquifer test at DRW-1. Completion logs for DRW-1 and the two deep observation piezometers were not generated by OHM.

3.0 AQUIFER TESTING PROCEDURES

OHM's written aquifer testing procedures are presented in Attachment A. A Baker representative was present at the site during the deep aquifer test to confirm that these procedures were properly followed. The deep pumping test was conducted by OHM between February 18 and 23, 1996. The shallow pumping test was conducted from February 24 to 28, 1996. OHM's field notes taken during the shallow and deep pumping tests are included as Attachment B.

3.1 Shallow Well Pumping Tests

OHM personnel conducted the shallow pumping test starting on Monday, February 24, 1996. Figure 2-1 shows the locations of SRW-1 and the observation wells and piezometers.

3.1.1 Test Configuration and Setup

According to the OHM aquifer test procedures and recovery well specifications, a submersible pump (Grundfos Model 5S) capable of pumping up to 7 gallons per minute (gpm) was installed into SRW-1 and set about 2 feet from the bottom of the well. For the purpose of the test, a pressure transducer was placed into SRW-1 a few feet above the pump. The available drawdown above the transducer was about 16 feet from the pre-pumping level.

The pressure transducer in SRW-1 was secured so that it could not move up or down in the well and was connected to a Hermit brand automatic datalogger (with 16 channels) along with the seven other transducers placed in nine observation points (from nearest to farthest: SP-1, SP-2, SP-3, 6GW-34, DRW-1, 6GW-1S, and 6GW-33). The dataloggers recorded the drawdown throughout the continuous-rate pumping test (both the drawdown and recovery phases).

DISTANCES FROM SRW-1 TO SHALLOW AQUIFER OBSERVATION POINTS

WELL ID	DISTANCE (in feet)
SP-2	13
SP-1	17
SP-3	98
6GW-34	99
DRW-1	285
6GW-1S	293
6GW-33	354

3.1.2 Shallow Step-Drawdown Test

OHM did not record the data from the step-drawdown test at SRW-1. The flow totalizer read 000. When the pump was turned on initially at 3:00 PM, the rate was 3 gpm and then decreased to 2.4 gpm after 40 minutes. At 2.4 gpm there was 3.6 feet of drawdown. Even with the valve fully opened, the flow rate could not be increased. Since the pump was rated at 7 gpm, it was suspected that the collapsible fire hose was impeding the flow of water from the well. This was confirmed when the pump was pulled from the well and placed in a 55-gallon drum: without the hose it was able to pump 6 gpm.

The following morning (Sunday, February 25, 1996), the fire hose was replaced by a 1" PVC line. The pump was turned on at 12:30 PM and the pump started pumping 6 gpm with the valve fully opened. After 30 minutes, the rate decreased to 5.0 - 5.5 gpm. After 55 minutes, the drawdown in the pumping well is 9.5 feet. Because the pump was pumping at its full capacity, it is not known whether the well could sustain a higher rate than 6 gpm if all of the available drawdown (at least 16 feet) were used.

After 60 minutes, the pump was turned off to allow the well to recover for two hours before starting the drawdown phase of the shallow continuous-rate test. No reading of the flow totalizer was recorded.

3.1.3 **Shallow Continuous-Rate Pumping Test**

The continuous-rate pumping test consisted of two phases: the first phase was drawdown (when the pump was turned on) and the second was recovery (when the pumped was subsequently turned off). The dataloggers were programmed to record water levels in all seven observation points at logarithmic intervals (more frequent at the beginning and less so as time passed) for both phases of the test. The drawdown data recorded by the dataloggers for the shallow test are presented in Attachment C. The recovery data recorded by the dataloggers are also presented in Attachment C.

3.1.3.1 <u>Shallow Drawdown Phase</u>

At 3:30 PM on Sunday, February 25, 1996, the pump in SRW-1 was turned on and pumped at a rate of about 5.5 gpm for four hours when (at 7:30 PM) it was discovered that the datalogger had lost the values for scale, linearity, and offset for the transducer in SRW-1. The pump was then turned off and the datalogger was re-programmed. No reading of the flow totalizer was recorded in the field log.

Two hours later (9:30 PM), the test was restarted at a rate of 5.5 gpm. After one hour, the drawdown in SRW-1 was 8.95 feet. Eventually the rate stabilized around 5 gpm and drawdown stabilized at about 10 feet. The test ran until 9:40 PM on Wednesday, February 28, 1996 when the pump was turned off for the start of the recovery phase.

Attachment C contains the data from the pumping (drawdown) phase of the shallow continuous-rate pumping test.

The flow totalizer was recorded for a portion of the shallow pumping test. For 11.5 hours the well produced 3,198 gallons for an average of 4.6 gpm. The estimate written in the log (5.09 gpm) is erroneous in that it did not take into account the "false starts." Unfortunately, no records of the flow totalizer exist at the start and the end of the shallow pumping phase, therefore, no volume of total pumpage or average rate throughout the test was possible.

3.1.3.2 Shallow Recovery Phase

After the pump was shut off, SRW-1 recovered to 6.49 feet of drawdown after 2.5 minutes. The datalogger recorded the recovery test overnight and for part of the next day (March 1, 1996) for a total length of 32 hours (1,920 minutes). Attachment C contains the data from the recovery phase of the shallow continuous-rate pumping test.

3.2 Deep Well Pumping Tests

3.2.1 Test Configuration and Setup

According to the OHM aquifer test procedures and recovery well specifications, a submersible pump (Grundfos Model 80S) capable of pumping up to 110 gallons per minute (gpm) was installed into DRW-1 and set about 2 feet from the bottom of the well. For the purpose of the test, a pressure transducer was placed into DRW-1 a few feet above the pump. The pressure transducer was secured so that it could not move up or down in the well and was connected to a Hermit brand automatic datalogger (with 16 channels) along with the nine other transducers placed in nine observation points (from nearest to farthest: 6GW-1DB, 6GW-1DA, 6GW-1D, 6GW-1S, DP-2, DP-1, SP-2, SRW-

1, and 6GW-15D). The dataloggers recorded the drawdown throughout the step-drawdown test and the continuous-rate pumping test (both the drawdown and recovery phases).

In addition to the five observation wells monitored automatically by the datalogger, one observation well (6GW-38D) was monitored manually. The table below shows the distances from the pumping well (DRW-1) to each observation well.

DISTANCES FROM DRW-1 TO DEEP AQUIFER OBSERVATION POINTS

WELL ID	DISTANCE (in feet)
6GW-1DB	38
6GW-1DA	42
6GW-1D	56
6GW-1S	77
DP-2	205
DP-1	213
SP-2	271
SRW-1	284
6GW-15D	377

3.2.2 Deep Step-Drawdown Test

The purpose of the step-drawdown test was to determine the optimal pumping rate at which drawdown in the pumping well is safely above the pump intake and the transducer and also at which the drawdown in the observation wells is maximized and easily measured. A step-drawdown test is performed by pumping the well at an initially low discharge rate for a period of time, observing the drawdown, and subsequently increasing the rate until an optimum pumping rate is determined. The step-drawdown test was performed on the morning of Monday, February 19, 1996 and was to consist of four 60-minute phases (steps): the initial rate was to be 50 gpm, the second was to be 70 gpm, the third 90 gpm, and the last 110 gpm.

The flow totalizer at the start of the step-drawdown test read 200 gallons. At 9:30 AM, the pump was turned on at the initial rate of 50 gpm. However, the sustainable flow rate from DRW-1 was much less than anticipated. The initial pumping rate (50 gpm) quickly produced drawdown to within 6 feet of the pump intake with cavitation (entrained air bubbles) around the pump. The pumping rate was quickly lowered (by manually closing the valve) to 29 gpm which eliminated the cavitation and stabilized the falling water level at about 2 feet above the pump intake. It was then decided that the well (as installed) could sustain no more than about 30 gpm with the available drawdown. At 9:45 AM (15 minutes of pumping), the pump was shut off and the water level allowed to recover for 15 minutes. The totalizer then read 1,495 gallons for a total of 1,295 gallons. However, this value may be inaccurate due to the cavitation and entrained air in the system during the initial startup.

At 10:00 AM, the pump was turned on again at 30 gpm and allowed to run for 90 minutes. At 11:30 AM, the water level had stabilized with a drawdown of 69 feet from the static level. This translates to a specific capacity (S_c) for DRW-1 of: Q/s = 30 gpm / 69 feet = 0.43 gpm/ft. By comparison to information on the well completion log for HP-651 (in Attachment C), the nearby water supply well produced 242 gpm with about the same amount of drawdown as in DRW-1 (S_c = 242 gpm / 69 ft = 3.5 gpm/ft). This is a very large difference (almost a full order of magnitude) and may be due to a number of factors including well design and the problems encountered (formation caving) during well installation. The screen length in DRW-1 is only 20 ft compared to about 70 ft (discontinuous) in HP-651. The shorter screen (and well depth) limits the available drawdown in DRW-1 and is probably another factor in the lower than expected well yield.

Attachment D presents the data from the deep step-drawdown test. After 90 minutes of pumping at 30 gpm, a maximum drawdown of 1.6 feet was generated in 6GW-1D at a distance of 56 feet from DRW-1. This was the closest observation well that was screened in a similar interval as DRW-1. It was determined that DRW-1 could produce no more than 30 gpm for a sustained yield. Because sufficiently measurable drawdown was generated during the step-drawdown test, the rate of 30 gpm was considered adequate for the pumping phase of the continuous rate test. After 90 minutes of pumping the flow totalizer read 4,437 gallons. A total of 2,942 gallons were pumped for an average of 32.7 gallons per minute.

3.2.3 **Deep Continuous-Rate Pumping Test**

The continuous-rate pumping test consisted of two phases: the first phase was drawdown (when the pump was turned on) and the second was recovery (when the pumped was subsequently turned off). The dataloggers were programmed to record water levels in all eleven wells at logarithmic intervals (more frequent at the beginning and less so as time passed) for both phases of the test. The drawdown data recorded by the dataloggers for the deep test are presented in Attachment D. The recovery data recorded by the dataloggers are also presented in Attachment D.

3.2.3.1 Deep Drawdown (Pumping) Phase

OHM personnel were at the site around the clock to conduct the continuous-rate pumping test on Tuesday, February 20, 1996 at the pre-determined rate of 30 gpm. The flow totalizer read 4,473 gallons before the pumping phase started. The datalogger started recording at 7:50AM when the pump was started. Because of the initally high head in the pumping well, the initial pumping rate was 50 to 56 gpm. This soon diminished as drawdown reduced the head inside the pumping well. Within minutes the rate stabilized to about 30 gpm with about 69 feet of drawdown in DRW-1.

After one hour, the rate was 29 gpm with about 70 feet of drawdown. After two hours the rate was 27 gpm and after 5 hours, the rate was 26 gpm which was the average rate for the remainder of the pumping phase of the test. Drawdown also stabilized at about 69 to 70 feet in the pumping well. After the first day, the rate was the same but the drawdown decreased to about 67 feet. Recharge from the preciptation events may have played a part in this recovery.

At the end of the pumping phase, the flow totalizer read 114,657 gallons. A total of 110,220 gallons was pumped during the 72-hour continuous-rate pumping test for an average flow rate of 25.5 gpm.

The pumping test generated a maximum of 2.27 feet of drawdown in the closest observation well (6GW-1D) at the end of the drawdown phase. All six of the deep observation points and piezometers responded during the pumping phase. The three shallow observation points also responded before being influenced by the recharge from precipitation that reversed the observed drawdown.

A contour map (Figure 3-1) of the resulting water table was generated during maximum drawdown (just before the recovery phase started). Because the well efficiency of the pumping wells is not 100%, the drawdown inside the pumping well is more than that in the formation just outside the well. For this reason, the actual drawdown in the formation near DRW-1 was estimated to reasonably match the actual conditions in the aquifer. An extrapolation of the regression line to the y-axis on the Distance Drawdown graph (Figure 3-2) gives a drawdown of about 4 feet at 10 feet from DRW-1; at 1 foot, the drawdown would be 2.27 feet more given the change per cycle. This indicates a drawdown of about 6.3 feet at one foot from DRW-1. This also indicates that the well efficiency of DRW-1 is less than 10% (6.3 ft / 69 ft = 0.091).

3.2.3.2 <u>Deep Recovery Phase</u>

The recovery phase of the continuous-rate test was started at 7:53 AM on Friday, February 23, 1996. The datalogger was set to record the recovery at logarithmic intervals and the recovery phase was recorded for 24 hours. The recovery data are presented in Attachment D. After 24 hours, the water levels in most of the affected observation wells recovered to within 0.13 feet or less of the pre-pumping conditions. The two exceptions were piezometers DP-1 and DP-2 which recovered only to 0.58 feet and 0.34 feet of the pre-pumping levels, respectively. This may be because of the temporary nature of the piezometers and their construction. However, the construction details for these wells and pizeometers cannot be verified because no well or piezometer completion logs were generated by OHM.

4.0 AQUIFER TESTING DATA EVALUATION

The recorded data were downloaded from the dataloggers into PC-compatible formatted files for use in spreadsheets and AqTeSolv (Aquifer Test Solver), an aquifer test data evaluation software package (version 2.0, Duffield, 1994). The data were then corrected as necessary (negative elevation values were converted to positive drawdown by OHM) and formatted as ASCII files for use in AqTeSolv.

Tables 4-1 and 4-2 present the results from the shallow and deep pumping tests, respectively, The data were evaluated by three different methods (unsteady drawdown, steady-state drawdown, and recovery). The tables list the minimum, average, and maximum values for permeability by well and by method. All data were corrected as necessary for unconfined conditions and for partial penetration within AqTeSolv.

The drawdown values for the pumping wells, SRW-1 and DRW-1, were not used in the calculations because of the well loss (due to friction) within the pumping wells. The recovery data for these pumping wells were used in the anaylses. The results of the distance-drawdown analyses are at the bottom of the table (in the average row). It should be stated here that the graphs of drawdown versus time in the attachments show the pumping rate to be 30 gpm. The software actually takes into account the entire series of pumping rates as the test progresses. The initial rate was 30 gpm which diminished over time until 26 gpm was the rate for the majority of the test.

The average permeability of the different methods used for the shallow pumping test was 1.68 ft/day (with a standard deviation of 0.65 ft/day). The average storativity value was 0.009 (unitless, with a standard deviation of 0.007). The average permeability for the deep pumping test was 4.90 ft/day (with a standard deviation of 1.69 ft/day). The average storativity value was 0.015 (unitless, with a standard deviation of 0.007).

4.1 Shallow Drawdown Phase Data Evaluation

The unsteady drawdown evaluation of the shallow data was performed using two equations: the Cooper-Jacob straight-line method corrected for unconfined drawdown (Cooper & Jacob, 1946; Kruseman & DeRidder, 1990) and the Theis curve-matching method corrected for unconfined drawdown (Theis, 1935; Kruseman & DeRidder, 1990). These two methods for analyzing unsteady flow to a well were performed within AqTeSolv. The steady-state pumping evaluation of the shallow data was performed using Jacob's distance-drawdown relationship using a spreadsheet to generate a least-squares regression line through the data (Jacob, 1950). The shallow recovery data were analyzed using the Theis method for recovery (Theis, 1935) also within AqTeSolv.

4.1.1 Unsteady Flow under Shallow Unconfined Conditions

Unsteady flow to a well occurs in the time interval after the pump is turned on in which the actual water table in the formation is changing (drawdown is increasing) with time. Two methods were used to evaluate the unsteady flow in the shallow pumping test. The two sets of columns in Table 4-1 labelled "Cooper-Jacob, Unconfined" and "Theis, Unconfined" present the results of the unsteady shallow pumping test data evaluation. The Cooper-Jacob method is a straight-line (semi-log) method and is usually easy to use and interpret. The analytical equation and the graphs used in the Cooper-Jacob method are presented in Attachment E. The Theis method is a curve-matching (log-log) method that requires somewhat more interpretation than the more intuitive straight-line method. The analytical equation and the graphs used in the Theis method are presented in Attachment F. Both methods usually assume the aquifer is confined, therefore the data were required to be adjusted to account for the unconfined nature of the surficial unit. The simple method by Kruseman and DeRidder was used within AqTeSolv for that purpose (Kruseman & DeRidder, 1990).

As shown in Table 4-1, the average permeability of the surficial unit according to the Cooper-Jacob method is 1.77 ft/day (with a standard deviation of 1.42 ft/day). The average storativity of the surficial unit by the same method was 0.004 (with a standard deviation of 0.003).

The average permeability from the Theis method was 1.39 ft/day (with a standard deviation of 1.15 ft/day). The average storativity was 0.006 (with a standard deviation of 0.004).

4.1.2 Steady-State Flow under Shallow Unconfined Conditions

Steady-state flow occurs after pumping has continued at a constant rate for some time when the drawdown stabilizes and reaches a dynamic equilibrium. The Jacob distance-drawdown method was used with the data collected after 72 hours (4,320 minutes) of pumping. This assumes that the drawdown had stabilized at that time. Figure 4-1 presents the drawdown as a function of the log of distance from the pumping well in the surficial unit. A least-squares linear regression was run through the data points from wells screened in the surficial unit. The transmissivity and storativity can be determined from the graph and the following equations for use with consistent units (Jacob, 1950):

$$T = (2.3\ Q) / (2\ \pi\ \Delta s)$$

and

$$S = (2.25\ T\ t) / r_o^2$$

For nonconsistent units where T is in ft^2/day and Q is in gpm, the equations are:

$$T = (70\ Q) / \ \Delta s$$

and

$$S = (T\ t) / (640\ r_o^2)$$

The discharge rate, Q, was 5 gpm (0.668 ft^3/min). The term Δs is the amount of drawdown in one cycle of log distance (1.13 feet). The regression line was extended to the x-axis so that the x-intercept (r_o) could be determined. The value of r_o was determined to be 350 feet from SRW-1. The value of time, t, at which the drawdowns in each well were plotted was 4,320 minutes.

The value of T from this method is 310 ft^2/day which corresponds to a permeability, (K = T/b) of 1.03 ft/day (with an aquifer thickness, b, of 300 feet -- no confining units were present at Site 82). The calculated value of storativity, S, was 0.017 (unitless).

4.2 Shallow Recovery Phase Data Evaluation

The analytical equation and the graphs used in the straight-line Theis Recovery evaluation of shallow pumping test data are presented in Attachment G. The values of permeability from the straight-line adaptation of the Theis method (Theis, 1935) had an average of 2.54 ft/day (with a standard deviation of 1.13 ft/day). These values are somewhat higher than those from the unsteady and steady drawdown methods. No estimation of storativity is possible with this method.

4.3 Deep Drawdown Phase Data Evaluation

The unsteady drawdown evaluation of the deep data was performed using three equations: the Neuman method (Neuman, 1974), the Cooper-Jacob straight-line method corrected for unconfined drawdown (Cooper & Jacob, 1946; Kruseman & DeRidder, 1990) and the Theis curve-matching method corrected for unconfined drawdown (Theis, 1935; Kruseman & DeRidder, 1990). These three methods for analyzing unsteady flow to a well were performed within AqTeSolv. The steady-state pumping evaluation of the deep pumping data was performed using Jacob's distance-drawdown relationship using a spreadsheet to generate a least-squares regression line through the data (Jacob, 1950). The deep recovery data were analyzed using the Theis method for recovery (Theis, 1935) also within AqTeSolv.

4.3.1 Unsteady Flow under Deep Unconfined Conditions

Many times in an unconfined pumping test, the effect of delayed gravity yield will be seen in the graph of drawdown versus time. This happens at some time into the test where the rate of drawdown apparently decreases, after which the drawdown rate increases again. This happens as the flow to the well from the formation changes from mostly horizontal to mostly vertical. After a while the flow again becomes mostly horizontal and the effect disappears. The Neuman method of graphical analysis takes this effect into account. Although this effect was not seen in all the wells at Site 82, in some wells it is suspected to have occurred to a slight degree.

Attachment H contains the analytical solution and the resulting graphs of the Neuman curve-matching procedure for the drawdown portion of the deep continuous-rate aquifer test. Using AqTeSolv, the graphed data were matched to curves generated by Neuman's method of analysis of drawdown in partially penetrating observation wells in unconfined aquifers (Neuman, 1974). As shown on these graphs, enough data points were generated to define the later portions (right side) of the curves.

The first column of hydraulic conductivity values in Table 4-2 was generated using Neuman's method for evaluating unconfined aquifers. The average value of horizontal hydraulic conductivity from Neuman's method is 5.01 ft/day (with a standard deviation of 1.75 ft/day). The average vertical hydraulic conductivity computed from this method is 1.71 ft/day (with a standard deviation of 1.33 ft/day).

The Neuman method is the only method to estimate a value for specific yield. Specific yield refers to the amount of water yielded to gravity drainage (as opposed to specific retention -- that which is retained under gravity drainage) and is expressed in a decimal form typically ranging from 0.01 to 0.5, similar to porosity. The average specific yield calculated with Neuman's method was 0.025 (with a standard deviation of 0.033).

Attachment I contains the analytical solution and the resulting graphs of the Cooper-Jacob straight line procedure for the drawdown portion of the deep continuous-rate aquifer test. The Cooper-Jacob straight-line method (adjusted for unconfined conditions) yielded an average permeability of 5.18 ft/day (with a standard deviation of 1.78 ft/day). The average storativity was 0.008 (with a standard deviation of 0.005).

Attachment J contains the analytical solution and the resulting graphs of the Theis curve-matching procedure for the drawdown portion of the deep continuous-rate aquifer test. The Theis method (adjusted for unconfined conditions) yielded an average permeability of 4.31 ft/day (with a standard deviation of 0.81 ft/day). The average storativity was 0.010 (with a standard deviation of 0.007).

4.3.2 **Steady-State Flow under Deep Unconfined Conditions**

Figure 4-2 presents the Jacob method of distance-drawdown analysis of steady-state flow. The data graphed are drawdown values after 72 hours (4,320 minutes) of pumping. The assumption is that drawdown had stabilized after this time. The wells screened in similar depth intervals as DRW-1 are the wells through which the least-squares regression line was drawn (6GW-1D, DP-1, DP-2 and 6GW-15D). The other wells were either much deeper (6GW-1DA and 6GW-1DB) or were much shallower (SRW-1 and SP-1) than the interval pumped by DRW-1.

The analytical equations are the same for this analysis as for the shallow test (discussed above). For nonconsistent units where T is in ft^2/day and Q is in gpm, the equation is:

$$T = (70\ Q) / \Delta s$$

and

$$S = (T\ t) / (640\ r_o^2)$$

The discharge rate, Q, was 26 gpm. The term Δs is the amount of drawdown in one cycle of log distance (2.27 feet). The regression line was extended to the x-axis so that the x-intercept (r_o) could be determined. The value of r_o was determined to be 550 feet from DRW-1. The value of time, t, at which the drawdowns in each well were plotted was 4,320 minutes.

The value of T from this method is 802 ft^2/day which corresponds to a permeability, ($K = T/b$) of 2.67 ft/day (with an aquifer thickness, b, of 300 feet -- no confining units were present at Site 82). The calculated value of storativity, S, was 0.018 (unitless).

4.4 Deep Recovery Phase Data Evaluation

The analytical equation and the graphs used in the straight-line Theis Recovery evaluation of deep pumping test data are presented in Attachment K. The values of permeability from the straight-line adaptation of the Theis method (Theis, 1935) had an average of 7.35 ft/day (with a standard deviation of 5.65 ft/day). No estimation of storativity is possible with this method.

The value for DP-1 is suspect because its graph (shown in Attachment K) shows two distinct slopes, both of which have very high permeabilities relative to the recharge values of K in other wells. However, even if it is eliminated from consideration, the average permeability by all methods changes from 4.97 ft/day to 4.50 ft/day. The reason for the suspect value may be due to the construction of the piezometer itself; however, the construction cannot be checked because no detailed construction logs were generated by OHM for the wells and piezometers that they installed.

4.5 Conclusion

The average horizontal permeability of the surificial unit in the vicinity of SRW-1 is 1.68 ft/day and its average storativity is 0.009.

The average horizontal permeability of the Castle Hayne in the vicinity of DRW-1 is 4.90 ft/day and its average storativity is 0.015.

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TABLES

TABLE 4-1
MCB CAMP LEJEUNE -- SITE 82 SHALLOW PUMPING TEST DATA EVALUATION

February 25-29, 1996

	Cooper-Jacob Unconfined		Theis Unconfined		Jacob Distance - Drawdown		Theis Recovery	AVERAGE OF ALL METHODS	
Well	Horizontal Permeability (ft/day)	Storativity (unitless)	Horizontal Permeability (ft/day)	Storativity (unitless)	Horizontal Permeability (ft/day)	Storativity (unitless)	Horizontal Permeability (ft/day)	Horizontal Permeability (ft/day)	Storativity (unitless)
SRW-1	NA	NA	NA	NA	NA	NA	1.47		
SP-2	0.62	0.002	0.55	0.003	NA	NA	1.57		
SP-1	1.10	0.005	0.92	0.007	NA	NA	1.91		
6GW-34	2.54	0.001	2.40	0.001	NA	NA	2.58		
SP-3	3.91	0.002	2.81	0.006	NA	NA	4.11		
DRW-1	0.69	0.010	0.25	0.011	NA	NA	NA		
6GW-1S	NA	NA	NA	NA	NA	NA	NA		
6GW-33	NA	NA	NA	NA	NA	NA	NA		
Minimum (*)	0.62	0.001	0.25	0.001	NA	NA	1.57		
Maximum (*)	3.91	0.010	2.81	0.011	NA	NA	4.11		
Average (*) (by method)	1.77	0.004	1.39	0.006	1.03	0.017	2.54	1.68	0.009
Standard Deviation	1.42	0.003	1.15	0.004	NA	NA	1.13	0.65	0.007

TABLE 4-2
MCB CAMP LEJEUNE -- SITE 82 DEEP PUMPING TEST DATA EVALUATION

February 18-24, 1996

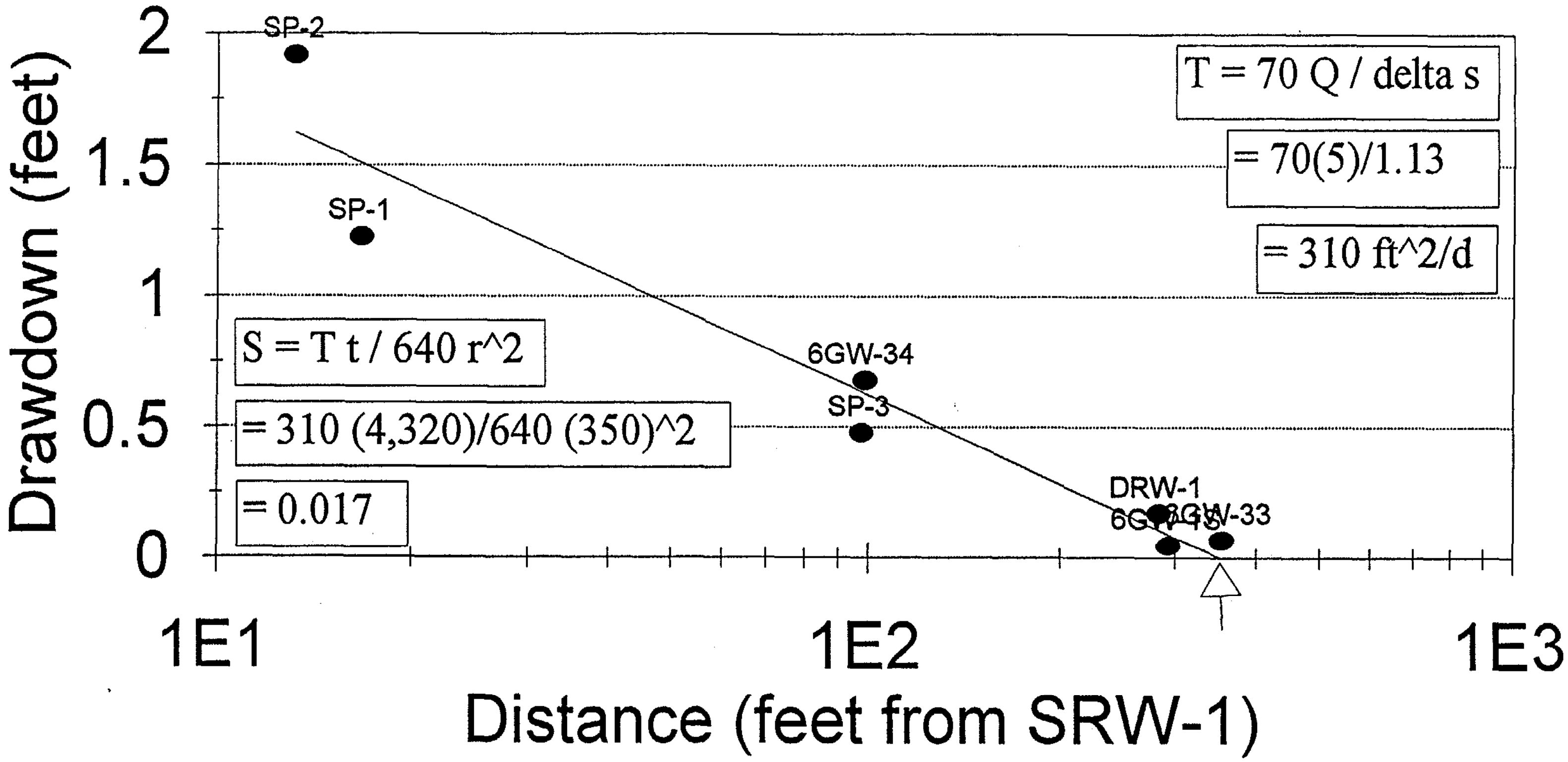
Well	Neuman Unconfined			Cooper-Jacob Unconfined		Theis Unconfined		Jacob Distance - Drawdown		Theis Recovery	AVERAGE OF ALL METHODS	
	Horizontal Permeability (ft/day)	Specific Yield (unitless)	Vertical Permeability (ft/day)	Horizontal Permeability (ft/day)	Storativity (unitless)	Horizontal Permeability (ft/day)	Storativity (unitless)	Horizontal Permeability (ft/day)	Storativity (unitless)	Horizontal Permeability (ft/day)	Horizontal Permeability (ft/day)	Storativity (unitless)
DRW-1	NA	NA	NA	NA	NA	NA	NA	NA	NA	3.09		
6GW-1DB	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA		
6GW-1DA	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA		
6GW-1D	7.16	0.001	1.2	4.46	0.001	4.11	0.001	NA	NA	5.47		
6GW-1S	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA		
DP-2	3.40	0.019	2.0	3.23	0.012	3.35	0.014	NA	NA	3.50		
DP-1	5.68	0.006	3.4	5.62	0.008	5.29	0.010	NA	NA	16.88		
SP-2	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA		
SRW-1	NA	NA	NA	NA	NA	NA	NA	NA	NA	NA		
6GW-15D	3.78	0.072	0.2	7.41	0.011	4.48	0.017	NA	NA	7.80		
Minimum (*)	3.40	0.001	0.22	3.23	0.001	3.35	0.001	NA	NA	3.50		
Maximum (*)	7.16	0.072	3.38	7.41	0.012	5.29	0.017	NA	NA	16.88		
Average (*) (by method)	5.01	0.025	1.71	5.18	0.008	4.31	0.010	2.67	0.018	7.35	4.90	0.015
Standard Deviation	1.75	0.033	1.33	1.78	0.005	0.81	0.007	NA	NA	5.65	1.69	0.007

* NOTE: Shallow observation points were not evaluated due to interference by recharge from precipitation.
Also, wells 6GW-1DA and 6GW-1DB were not evaluated due to vertical distance to pumping well >> horizontal distance.

FIGURES

Drawdown vs. Distance

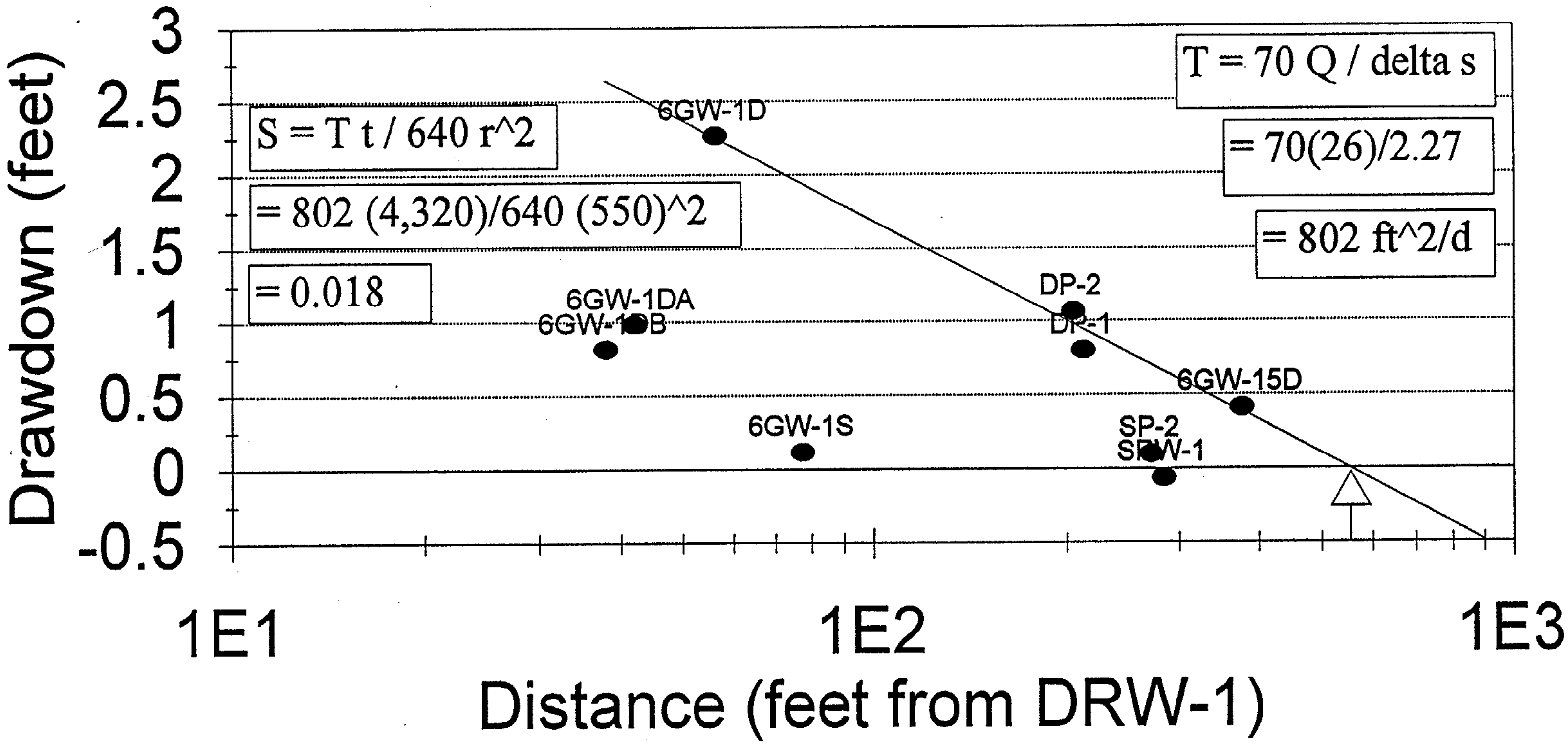
MCB Camp Lejeune -- Site 82



Drawdown after 4,320 minutes of pumping.

Drawdown vs. Distance

MCB Camp Lejeune -- Site 82



Drawdown after 4,320 minutes of pumping.

Attachment A – OHM Aquifer Testing Procedures

OHM Remediation
Services Corp.

AQUIFER TEST WORK PLAN
SITE 82
MCB CAMP LEJEUNE, NORTH CAROLINA

Prepared for:

DEPARTMENT OF THE NAVY
Contract No. N62470-93-D-3032
Delivery Order 0015

Prepared by

OHM Remediation Services Corp.
Norcross, Georgia

for

John P. Franz, P.E.
Program Manager

James A. Dunn, Jr., P.E.
Project Manager

January 1996

OHM Project No. 16032

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1.0 INTRODUCTION

The following aquifer testing program is presented in outline or macro format to allow for field modification as necessary to meet varying conditions. The general parameters presented herein are a result of meetings and discussions with Baker Environmental, LANTDIV, and OHM technical personnel. We anticipate conducting this test program immediately following the Christmas holidays, i.e., commencing the first week of January 1996. The new piezometer installation will occur prior to Christmas 1995.

2.0 *MONITORING WELL SAMPLING*

In order to properly evaluate the effectiveness of the treatment system, it is recommended that a complete round of monitoring well sampling be conducted prior to the aquifer test. This timing will allow for "baseline" sampling, i.e., prior to any activities which could influence static conditions in the aquifer.

2.1 INITIAL SAMPLING

In consultation with Baker Environmental, we have jointly developed the following lists of monitoring wells which are proposed to be sampled. Subsequent sampling events to be conducted to measure the effectiveness of the treatment system will involved fewer wells as indicated below.

Shallow Wells	**Deep Wells**
6 GW 1S	6 GW 1D
6 GW 33	6 GW 1DA
6 GW 34	6 GW 1DB
82 MW 3	6 GW 27D
6 GW 28S	6 GW 27DA
	6 GW 28D
SRW - 1	DRW - 1
6 GW 32	6 GW 40DW
	6 GW 40DWA
82 MW 2	
82 MW 1	6 GW 38D
82 MW 30	6 GW 15D
6 MW 3S	6 GW 30D
6 GW 3	6 GW 37D
6 GW 26	6 MW 3D
6 GW 30S	6 GW 2D
6 GW 2S	

Analytical testing to be performed on the groundwater samples will include EPA Method 8021 or 601/602 for volatile organic compounds (VOCs) and EPA Methods 6010/7060/7421/7470 or 200.7/2062/239.2/245.1 for target analyte metals (total). Results will be compared against the remediation goals set in the Record of Decision and indicated in the following table.

Remediation Goals for OU No. 2 - Groundwater Remediation

Media	Contaminant of Concern	Remediation Goal	Unit	Basis
Ground-water	1,2-Dichloroethane	0.38	ug/L	NCWQS
	Trans-1,2Dichloroethene	70	ug/L	NCWQS
	Ethylbenzene	29	ug/L	NCWQS
	Tetrachloroethene	0.7	ug/L	NCWQS
	Trichloroethene	2.8	ug/L	NCWQS
	Vinyl Chloride	0.015	ug/L	NCWQS
	Arsenic	50	ug/L	NCWQS
	Barium	1,000	ug/L	NCWQS
	Beryllium	4	ug/L	MCL
	Chromium	50	ug/L	NCWQS
	Lead	15	ug/L	MCL
	Manganese	50	ug/L	NCWQS
	Mercury	1.1	ug/L	NCWQS
	Vanadium	80	ug/L	Health Advisory

NCWQS = North Carolina Water Quality Standard
MCL = Maximum Contaminant Level

2.2 GROUNDWATER SAMPLE COLLECTION

The monitoring wells will be sampled via low-flow methods. Low-flow is defined as a flow rate similar to the ambient flow rate in the screened formation.

A peristaltic pump will be used to purge the wells and collect the samples. VOC loss through suction degassing is expected to be insignificant due to the very slow flow rates to be used. The procedure for collecting groundwater samples has been provided by Baker. Further details are provided in the amendment to the Field Sampling Plan appended to this document.

1. The protective casing (for existing wells) will be unlocked, the well cap will be removed, and escaping gases will be measured at the well head using a PID or FID. This will determine the need for respiratory protection.

2. The well will be allowed to equilibrate to atmospheric pressure, in the event that a vent hole was not installed in the well.

3. The static water level will be measured. The total depth of the well will not be measured, as not to stir up any sediment. The total depth will be obtained from boring logs. The water volume in the well will then be calculated.

4. The sampling device intake (virgin, 1/4 inch ID Teflon tubing) will be slowly lowered until the bottom end is 2 to 3 feet below the top of water. Based on historical water levels, this depth will be a point within the screened interval. Next, the water level probe will be placed into the well, just below the surface of the water.

5. Purging will then begin. The discharge rate will be measured using a stopwatch and calibrated container. The flow rate will be adjusted to ambient flow conditions (i.e., no drawdown is observed in the well). Flow rates of less than 1 liter per minute (L/min) are expected.

6. The Water Quality Parameters (WQPs), including dissolved oxygen, turbidity, temperature, pH, and specific conductance will be measured frequently (e.g., every 2 minutes).

7. Purging will be complete when three successive WQP readings have stabilized within 10 percent, or there is no further discernable upward or downward trend. Low values, certain WQPs (such as turbidity and dissolved oxygen) may vary by more than 10 percent, but have reached a stable plateau.

8. Upon WQP stabilization, groundwater samples will be collected. Samples for VOC analysis will be collected first, followed by total metals. Sample bottles will be labeled prior to sample collection.

9. Replace the Teflon and silicon pump tubing between wells.

10. The sample jars will be stored in a cooler with ice until laboratory shipment. Samples must be shipped within 24 hours of collection.

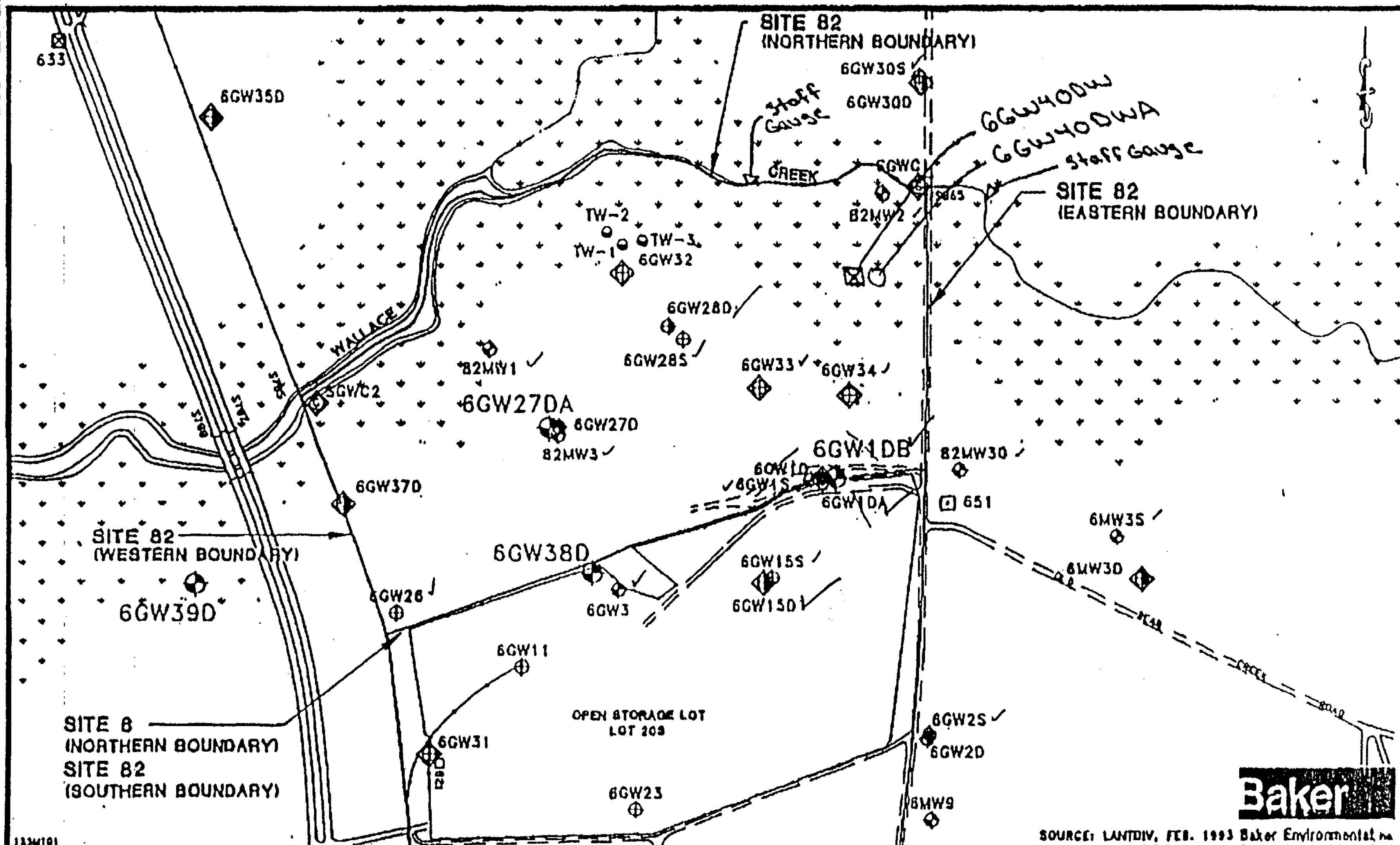
2.3 QUALITY CONTROL/QUALITY ASSURANCE PROGRAM

Three types of field quality assurance/quality control samples will be submitted to the laboratory: trip blanks, field blanks, and field duplicates. Since dedicated tubing is used for each well, no equipment rinsates will be required. The results from the field quality control samples will be used to determine the overall quality of the data.

2.4 QUARTERLY SAMPLING

Monitoring wells recommended to be sampled on a quarter basis to measure effectiveness of the treatment system include the following:

Shallow Wells	Deep Wells
6 GW 15	6 GW 1D
6 GW 33	6 GW 1DA
6 GW 34	6 GW 1DB
82 GW 3	6 GW 27D
6 GW 28S	6 GW 27DA
	6 GW 40DW
	6 GW 40 DWA



LEGEND

- EXISTING SHALLOW MONITORING WELL
- PHASE I SHALLOW MONITORING WELL INSTALLED BY BAKER ENVIRONMENTAL, Inc., SEPT.- OCT, 1992
- PHASE I DEEP MONITORING WELL INSTALLED BY BAKER ENVIRONMENTAL, Inc., SEPT.- NOV, 1992
- PHASE I STAFF GAUGE INSTALLED BY BAKER ENVIRONMENTAL, Inc., OCT, 1992
- PHASE II SHALLOW MONITORING WELL INSTALLED BY BAKER ENVIRONMENTAL, Inc., MARCH-APRIL 1993
- PHASE II DEEP MONITORING WELL INSTALLED BY BAKER ENVIRONMENTAL, Inc., MARCH-APRIL 1993
- PHASE II STAFF GAUGE INSTALLED BY BAKER ENVIRONMENTAL, Inc., DEC. 1992
- TEMPORARY WELL INSTALLED BY BAKER ENVIRONMENTAL, Inc., MARCH 1993

500 0 250 500 1000

1 inch = 500 ft.

FIGURE 1
DEEP MONITORING WELL LOCATIONS
PRE-DESIGN STUDY CTO-0133

MARINE CORPS BASE, CAMP LEJEUNE
NORTH CAROLINA

3.0 PIEZOMETER INSTALLATION

To properly measure the hydrogeologic properties of the aquifer, it has been deemed necessary to install piezometers which will be monitored during the aquifer test. The locations for these new 1-inch piezometers are indicated of drawings CD-10A and CD-10 and are described as follows.

3.1 SHALLOW AQUIFER

The aquifer test of the shallow aquifer will be conducted through well SRW-1. Three new 1-inch piezometers designated SP-1, SP-2, and SP-3 will be installed 20 feet north, 10 feet south, and 100 feet east of well SRW-1. Depth of the piezometers and locations of well screens will mirror SRW-1.

3.2 DEEP AQUIFER

The aquifer test of the deep aquifer will be conducted through well DRW-1. Two new 1-inch piezometers designated DP-1 and DP-2 will be installed 200 feet west of DRW-1 and midway between DRW-1 and 6 GW 15D. Depth of the piezometers and locations of the well screen will mirror DRW-1.

4.0 AQUIFER TEST

The aquifer test program for both the shallow and the deep aquifer testing will include three phases. The initial phase will be a stepped drawdown test. The second phase will be a 72-hour pumping test. The final phase will be a recovery test.

4.1 SHALLOW AQUIFER TEST

As indicated earlier, the pumping well for the shallow aquifer test will be SRW-1. Wells to be monitored during the test are SRW-1, SP-1, SP-2, SP-3, 6 GW 34, 6 GW 33, 6 GW 1S, and DRW-1. For ease of identification, each of these wells has been italicized on the attached drawings. A Hermit datalogger with 16 port capacity will be used to continuously monitor the water levels in each of the eight wells. Produced fluids will be piped to the new groundwater treatment plant for treatment and subsequent discharge into Wallace Creek.

4.2 DEEP AQUIFER TEST

As indicated earlier, the pumping well for the deep aquifer test will be DRW-1. Wells to be monitored during the test are DRW-1, DP-1, DP-2, 6 GW 15D, 6 GW 38D, 6 GW 1S, 6 GW 1D, 6 GW 1DA, 6 GW 1DB, SRW-1, and SP-2. For ease of identification, each of these wells has been italicized on the attached drawings. A Hermit datalogger with 16 port capacity will be used to continuously monitor the water levels in each of the 11 wells. Produced fluids will be piped to the new groundwater treatment plant for treatment and subsequent discharge into Wallace Creek.

4.3 DETAILED DESIGN

The purpose of aquifer test to be performed at Site 82, MCB Camp Lejeune is to collect data on the reaction of the aquifer(s) in various observation wells in response to pumping from shallow and deep recovery wells. This data will be analyzed by Baker Environmental.

Prior to the aquifer tests, groundwater samples will be collected from the monitoring and recovery wells at the site. These will be analyzed to identify the spatial distribution and concentration gradients of dissolved contaminants. The procedures for this baseline sampling are described in the Field Sampling Plan, which is attached to this document as Appendix A.

This detailed design outlines the specific equipment, procedures, and data interpretation to be performed during the testing of the shallow and deep aquifers. Wherever possible, the equipment, flow rates and durations are identified quantitatively. Because no previous pumping tests have been performed at the site, the test parameters are based on inferences from available information. Given the complex and heterogenous nature of aquifers, initial field results may indicate that some modification of these parameters is necessary during testing.

4.3.1 Equipment

A) Pumps, Flow Meters, Flow Rate Controls

1) General: The two pumping wells (DRW-1 and SRW-1) will be supplied with electric pumps, flow meters covering the full range of expected flow rates, and valves for adjusting the flow rates. The power source is the base electrical network. All of this equipment will be installed, wired, tested and operational prior to the aquifer tests.
2) Shallow aquifer: The pump is a Grundfos Model 5S, with operational flow rates ranging from 1.2 to 7 gallons per minute (gpm).
3) Deep aquifer: The pump is a Grundfos Model 80S, with operational flow rates ranging from 48 to 110 gpm.

B) Data Logger, Pressure Transducers, Cables, and Lap Top Computer

1) Data Logger: Because a maximum of 11 wells (including the pumping well) will be monitored at any one time, a Hermit Model SE2000 with 16 channels will be employed. During the tests, the data logger will be located in the well house of the pumping well.
2) The choice of pressure transducers is determined by the presence or absence of contaminants, the range of pressures expected, and the diameter of the wells.
   a) Because contaminants are expected to be present in at least some of the wells, the transducer will be supplied with teflon sheathes rather than polyurethane sheathes.
   b) Each transducer will be either a standard range or special range model. The special range will be required in the pumping wells. The standard range will be adequate for the majority of the monitoring wells.
   c) A standard pressure transducer has an outside diameter of an inch. The piezometers which are used for monitoring will require be specialized transducers with reduced outside diameters.
3) Cables: The pressure transducers come with cable assemblies ranging from 150 feet to 500 feet in length. Additional connectors approximately 350 feet long each are available for those wells which are further from the data logger.
4) Lap top computer: During testing, data will be downloaded at least once every 24 hours from the data logger. For this purpose, an IBM compatible Model 486 (or equivalent) will be utilized. The specific software program for downloading the data will be supplied with the data logger.

C) Wells: In order to properly interpret the test data from pumping and observation wells, several details of well construction will be utilized. These are:

1) Total depth
2) Screened interval
3) Well diameter and slot size
4) Annular space diameter
5) Method of completion
6) Elevation of top of filter pack, bentonite seal and grout
7) Surveyed elevation of top of casing
8) Elevation of upper and lower boundaries of the aquifer being monitored
9) (in pumping wells) The depth at which the pump is installed
10) The length of the pump.

D) Miscellaneous

1) Rain gauge: Recharge during the test period can affect aquifer test results. To quantify this effect, a rain gauge will be mounted outside the well house of the pumping well during each test to measure precipitation.
2) Barometric pressure gauge: Fluctuations in atmospheric pressure can also affect aquifer test results. To quantify this effect, a barometric pressure gauge will be mounted outside the well house of the pumping well during each test to measure changes in atmospheric pressure.
3) Water level indicator: This instrument will be used to confirm the proper functioning of the pressure transducers and to measure water levels in the distant well selected for observation of regional trends (see II-A-5 below).
4) Water treatment and disposal: The tests are being performed in portions of the aquifers where organic contaminants have been reported. A water treatment system is under construction and will be operational before the aquifer tests are performed. All effluent from the tests will be piped to this system for treatment, followed by discharge to Wallace Creek.

4.3.2 Field Procedures

A) General Procedures

1) Baseline: For all aquifer tests, a baseline measurement of conditions will be collected just prior to beginning the test.
2) Recovery: For all aquifer tests, the aquifer will be allowed to recover following the completion of the test. The recovery period will be a maximum of 24 hours for this test.
3) Precipitation: The rain gauge will be observed and amount of precipitation (if any) recorded once every 24 hours. If heavy rainfall events take place during testing, this frequency will be increased to once every 6 hours.
4) Atmospheric pressure: The barometric pressure gauge will be observed once every hour. Any changes in atmospheric pressure will be recorded.
5) Regional trend: changes in the elevation of the aquifer's potentiometric surface may occur due to natural causes during the tests. Therefore, for each test a well will be selected which is distant enough from the pumping well that no pumping influence would reasonably be expected to occur. During the test this well will be periodically monitored with a water level indicator to quantify any regional trends.
6) Potential interference from the active soil vapor extraction (SVE) system: The shallow aquifer test is taking place adjacent to and in the zone of influence of an active SVE system. The operation of this system could have unpredictable effects upon the results of the tests. Therefore, the SVE system will need to be shut down while the tests are being run and for a week before the tests begin (to allow for equilibration to static conditions).

B) Step drawdown tests

1) Goal: The goal of the step drawdown tests is to determine that maximum sustainable pumping rate in each pumping well which will maintain the maximum sustainable drawdown during the continuous aquifer test. The maximum sustainable drawdown is

the depth to the pump inlet minus a safety factor to ensure that the pump remains sufficiently submerged for proper operation.

2) General procedures: For each pumping rate, the drawdown in the pumping well is monitored until it approaches equilibrium closely enough for steady state conditions to be inferred. The time required to achieve this condition is dependent on both aquifer and well parameters, and cannot be predicted with precision in advance. Based on available information, it is expected that 1 to 2 hours will be sufficient for each pumping rate in each well at this site.
3) Deep aquifer: As described above, a Grundfos Model 80S pump will be used for the tests in DRW-1. This pump has a operational range from 48 to 110 gpm. Given the flow rates expected from the deep aquifer, it is anticipated that the step drawdown intervals will be 50, 70, 90 and 110 gpm. Prior to the step drawdown test, the pressure data recording equipment will be installed. During the step drawdown test this equipment will be operated and monitored to ensure it is performing properly prior to the continuous test.
4) Shallow aquifer: As described above, a Grundfos Model 5S pump will be used for the tests in SRW-1. This pump has an operational range from 1.7 to 7 gpm. Given the flow rates expected from the shallow aquifer, it is anticipated that the step drawdown intervals will be 2, 3, 4 and 5 gpm. Prior to the step drawdown test, the pressure data recording equipment will be installed. During the step drawdown test this equipment will be operated and monitored to ensure it is performing properly prior to the continuous test.

C) Continuous aquifer tests
1) Goal: The goal of the continuous aquifer tests is to determine aquifer parameters so that the optimal location(s) and construction details can be specified for additional recovery wells to capture contaminated groundwater.
2) Deep aquifer: The test will be run continuously for a period of 72 hours at the maximum sustainable pumping rate as determined by the step drawdown test. The observation wells will be DRW-1, DP-1, DP-2, 6 GW 15D, 6 GW 38D, 6 GW 1S, 6 GW 1D, 6 GW 1DA, 6 GW 1DB, SRW-1 and SP-2. The regional trend observation well will be 6 GW39D. In addition to continuous recording by the data logger, manual readings will be taken from select wells with a water level indicator to confirm proper operation of the data collection equipment. The intervals for these readings will be as follows:
   a) First 10 minutes of the test: every 2 minutes
   b) Next 30 minutes of the test: every 5 minutes
   c) For the remainder of the test: every 10 minutes, or as deemed appropriate by the senior hydrogeologist on site.
3) Shallow aquifer: The test will be run continuously for a period of 72 hours at the maximum sustainable pumping rate as determined by the step drawdown test. The observation wells will be SRW-1, SP-1, SP-2, SP-3, 6 GW 34, 6 GW 33, 6 GW 1S, and DRW-1. The regional trend observation well will be 6 GW 28S. Manual water level readings will be taken on the same schedule as shown above for the deep aquifer test.

5.0 REPORT OF FINDINGS

During the aquifer testing period, a Senior Hydrogeologist will be onsite to provide overall direction and guidance to the hydrogeologists performing the test. This individual will be evaluating the data on a daily basis to be in a position to make recommendations on future well placement locations as soon as practicable. It is recommended and encouraged that Baker Environmental, as designer of record, and LANTDIV, as owner, have representatives available onsite during the aquifer test program to provide input and concurrence with the recommendations of our Senior Hydrogeologist.

5.1 DATA INTERPRETATION

Preliminary examination of the data for indications of aquifer properties will occur daily during the field testing. The data which is downloaded from the data logger into the laptop computer will be copied on disks which will be made available to LANTDIV and Baker Environmental. Upon completion of the field tests, Baker will perform a detailed analysis of the data.

5.2 RECOMMENDATIONS

Baker Environmental will present the data interpretation and make recommendations regarding alternations to the groundwater recovery system design. OHM will review the interpretation and recommendations to the extent directed by LANTDIV.

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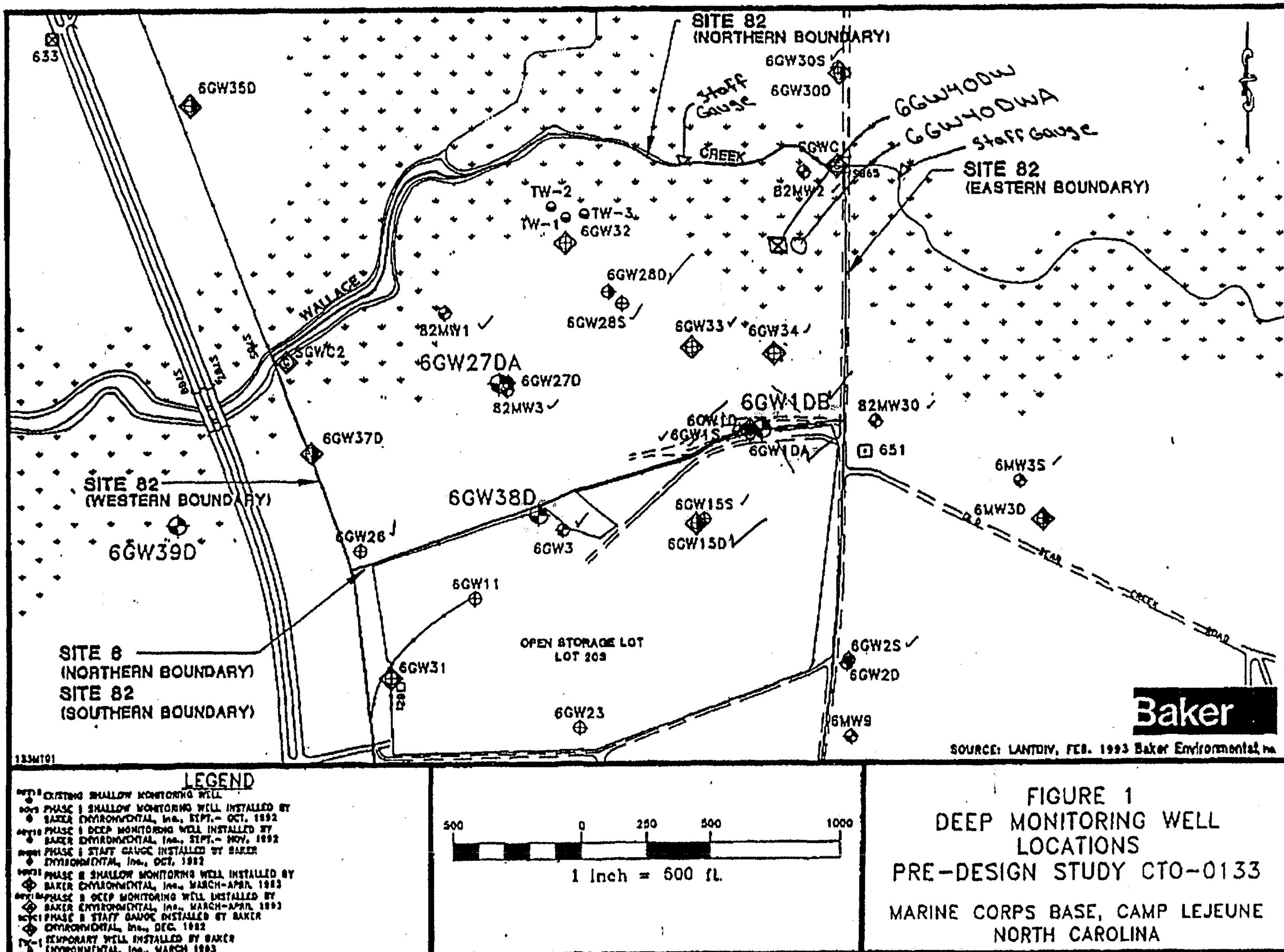

FIGURE 1
DEEP MONITORING WELL LOCATIONS
PRE-DESIGN STUDY CTO-0133

MARINE CORPS BASE, CAMP LEJEUNE
NORTH CAROLINA

APPENDIX A

ADDENDUM TO THE SAMPLING AND ANALYSIS PLAN

OHM Remediation
Services Corp.

Addendum to the Sampling and Analysis Plan for Soil and Groundwater Remediation Operable Unit No. 2 MCB Camp Lejeune, North Carolina

For Groundwater Collection at Site 82, Pre-Aquifer Test

January 1996

OHM Project No. 16032

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1.0 INTRODUCTION

Baseline sampling will be performed before the aquifer test to determine levels of target compounds in the groundwater along with other specific analytical tests that will be required for monitoring water quality. In addition water levels will be required on all wells for aquifer test background. Approximately 15 shallow and 15 deep well will be sampled. Carl Pampel (Project Chemist) will be on-site for the first week of the sampling effort to assist in training of personnel and trouble shooting any technical difficulties that arise.

2.0 DATA QUALITY OBJECTIVES

Specific remediation goals for OU No. 2 groundwater have been determined. Analytical methodology has been selected that will meet or surpass all remediation goal detection level for the final cleanup except for vinyl chloride. The detection goals for vinyl chloride are below all EPA analytical methodology and the best available detection limits will be employed. Table 2.1 summarizes the remediation goals and the additional water quality analysis for the project. Data quality parameter goals are: 50% RPD for field collocated duplicates, 20-150% recovery for MS/MSDs with 50% RPD, 30-150% LCS recovery, and 90% data completeness.

Table 2.1 Remediation Goals and Water Quality Parameters

Contaminant of Concern	Method Number	Remediation Goal ug/l	Laboratory Required Reporting Limit ug/l
1,2-Dichloroethane	8021	0.38	0.2
Trans-1,2-Dichloroethene	8021	70	2
Ethylbenzene	8021	29	2
Tetrachloroethene	8021	0.7	0.4
Trichloroethene	8021	2.8	1.5
Vinyl Chloride	8021	0.015	<1.0
Arsenic	7060	50	15
Barium	6010	1000	50
Beryllium	6010	4	2
Chromium	6010	50	15
Lead	7421	15	5
Manganese	6010	50	15
Mercury	7470	1.1	0.2
Vanadium	6010	80	20
Dissolved Oxygen (field test)	SM-4500-0 G (or equivalent)	NA	NA
pH (field test)	9040	NA	NA
BOD (5 day)	405.1	NA	NA
COD	410.4	NA	NA
Turbidity	180.1	NA	NA
Alkalinity	305.1	NA	NA
Hardness	130.2	NA	NA

3.0 WELL LEVEL DETERMINATION AND GROUNDWATER SAMPLE COLLECTION

Also refer to section 2.2 of the Work Plan.

Upon arriving at Site 82, the location of all wells to be sampled will be determined. As each is located, its pressure cap will be removed to allow equilibration. As soon as all wells are located, each will be gauged to determine the depth to water from top of casing (to the nearest 1/100 of a foot) using a water level indicator. This depth plus the date and time of gauging will be recorded.

The data will be used to determine the volume of water in the well casing. With the exception of the first well gauged, the pressure cap will be reinserted as soon as gauging is complete. At the conclusion of the survey the first well measured will be gauged and recorded again. This will allow any regional trends to be observed and appropriate corrections made. The first well will then be recapped.

The wells will be purged and sampled using low flow methods. A peristaltic pump with Teflon tubing will be used to purge and collect the required samples. After wells are gauged they will be purged using the peristaltic pumps at a flow that will not cause draw down of the well water level. It is expected that the flow rates will be less than 1 liter per minute. The pH and DO readings will be monitored and the well purged until stable readings are obtained. At least one well volume will be purged. After the well has been purged and the purge water collected the metals sample bottle will filled through the pump. COD, BOD, pH, Turbidity, alkalinity, hardness, and dissolved oxygen (DO) will be sampled through the pump into the appropriate container. DO and pH measurements will be conducted in the field following the instrumentation methods. The tubing before the pump will be capped with a finger to trap the water in the tubing. The tubing will be slowly pulled from the well and the water drained into the two VOA containers. All sample containers will already contain the correct preservatives. New Teflon and pump tubing will be used at each well.

All wells will require the metals and volatile analysis. Wells 6GW33, SRW-1, 82MW30, 6GW27D, DRW-1 and 6GW15D will require DO, pH, BOD, COD, turbidity, alkalinity and hardness in addition to the metals and volatiles. A listing of the shallow and deep wells can be found in section 2.1 of the Work Plan.

Table 3.1 summarizes the estimated required sample bottles and preservatives along with holding times for each method. Some analysis can be combined into the same containers.

Table 3.1 Analytical Summary

Analysis	Method Number	Preservatives	Containers	Holding Times
VOCs	8021	HCL<2/Cool 4 C	2ea 40ml VOA vials	14 days
Metals Hardness	6010/7000	Nitric<2	1ea 500ml plastic	6 months (Hg 28 days)
Alkalinity	310.1	Cool 4 C	1ea 500ml plastic	14 days
COD	410.4	Sulfuric<2/Cool 4 C	1ea 250ml plastic	28 days
pH	9040	Cool 4 C	1ea 250ml plastic	Immed.
Turbidity	180.1	Cool 4 C	1ea 500ml plastic	48 hours
BOD	405.1	Cool 4 C	1ea 1 liter plastic	48 hours

4.0 *QA/QC SAMPLES*

NFESC level C data reporting will be required for this project. One field blank will be required for each water source that is used in equipment deacon if needed. Rinsate blanks will not be required since the sampling equipment is not cleaned between sample points and new materials are used. Field collocated duplicates will be required at 10% to meet level C reporting. Trip blanks will accompany each cooler that contains volatile analysis. This blank will be sent from the laboratory and returned with the samples back to the laboratory and analyzed for volatiles only.

5.0 SAMPLE IDENTIFICATION

All wells are already numbered. This number with the date sampled will become the sample designation. Field duplicates will be dented with a D at the end of the sample number. For example: 6GW15-010496 for the sample and 6GW15-010496D for the duplicate.

OHM Remediation
Services Corp.
A Subsidiary of OHM Corporation

January 6, 1996

Cheryl Hansen/ROICC
Naval Facilities Engineering Command
1005 Michael Road
Camp Lejeune, NC 28540

Re: Contract N62470-93-D-3032; Delivery Order 0015
MCB Camp Lejeune, NC
OHM Project No. 16032
Aquifer Test Work Plan

Dear Ms. Hansen:

Enclosed herewith please find three copies each of the subject document which has been revised to reflect comments received from LANTDIV.

Very truly yours,
OHM Remediation Services Corp.

James A. Dunn, Jr., P.E.
Senior Project Manager

/mja

Enclosures

pc: Lance Laughmiller - Code 18233 (1)
Neal Paul - IRD/EMD w/enclosure (2)
Matt Bartman - Baker Environmental w/enclosure (1)
Randy Smith - OHM w/enclosure (5)
Patrick Watters - NCDEHNR w/enclosure (1)
John Franz - OHM w/enclosure (1)
Jerry Haste - NAVFACENGCOM - w/enclosure (1)
Gena Townsend - EPA Region IV - w/enclosure (1)
OHM Project File/16032 w/enclosure (1)
Dwayne Currie - OHM w/enclosure (1)

2/16/96

1)

1400 Unloading equipment and check it against
PO and packing list:
Troll w/ comm cables
Extension cable - 400' = IIII.
Hermit 16 Channel Data logger w/ comm cables
Pressure Transducer 20 psi - IIII
Pressure Transducer 10 PSI - IIII
Pressure Transducer 50 PSI
Pressure Transducer 30 PSI

Missing:
Extension cable 400' - 1 - Found
Pressure Trans - 20 PSI 1 Found.
Pressure Trans - 10 PSI 1 - Delivered - 2/17/96

2/17/96

8:30 G. Crowe arrives on site to string cable for transducer and program data logger for pumping test

9:00 Begin laying cable for transducers

1200 Lunch Break

1230 Return to lay cables

1500 Begin taking waterlevels so a depth to place the transducers can be determined

	TOC	TOS Protector
6GW38D	19.53/	19.65/
6GW15D	13.31	13.43
DP2	16.84	NA
6GW1DB	21.08	21.31
6GW1DA	20.84	21.61
6GW1D	21.03	21.29
6GW1S	17.62	17.85
DP1	24.55	NA
SP~~2~~ Gwc	17.22	NA
SRW	~~16.62~~ Gwc	16.62

1630 Return to office trailer to go over instructions for Troll data logger.

1800 off site

2/16/96

1)

1400 Unloading equipment and check it against
PO and packing list:
Troll w/ comm cables
Extension cable - 400' = IIII
Hermit 16 Channel Data logger w/ comm cables
Pressure Transducer 20 PSI - IIII
Pressure Transducer 10 PSI - IIII
Pressure Transducer 50 PSI
Pressure Transducer 30 PSI

Missing:
Extension cable 400' - 1 - Found
Pressure Trans - 20 PSI 1 Found
Pressure Trans - 10 PSI 1 - Delivered - 2/17/96

2/17/96

8:30 G. Crowe arrives on site to string cable for transducer and program data logger for pumping test

9:00 Begin laying cable for transducers

1200 Lunch Break

1230 Return to lay cables

1500 Begin taking water levels so a depth to place the transducers can be determined

	TOC	TOS Protector
6GW 38D	19.53	19.69
6GW 15D	13.31	13.43
DP2	16.84	NA
6GW1DB	21.08	21.31
6GW1DA	20.84	21.61
6GW1D	21.03	21.29
6GW1S	17.62	17.85
DP1	24.55	NA
SP~~1~~2 Gwc	17.22	NA
SRW	~~16.62~~ Gwc	16.62

1630 Return to office trailer to go over instructions for Troll data logger.

1800 Off site

1-23-95 Atlanta GAS & LIGHT Proj # 17312

0800 CREW MEETS AT SHOP TO MOBE TO AGL.
(M. BISHOP, E. ANDERSON, K. PHILLIPS)
0840 E ANDERSON LEAVES w/ 2 SAMPLES (SOIL)
FROM 1-22-95 TO DELIVER TO KIBER.
MILLEAGE (102561)
1015 E. ANDERSON ARRIVES ON SITE. (MILLEAGE 102611)
(915) BISHOP & PHILLIPS ARRIVE ON SITE.
- AGL CREW ALREADY ON SITE AND EXCAVITING
AREA OF NEW PIPE PUNCTURE.
- BISHOP MEETS w/ TERRY (AGL) AND IS INFORMED
THAT

Discard
Thomas A McIlroy
2/18/96

Proj. No. 16032 Camp Lejeune Aquifer Tests WBS 0293001 2/18/96

1300 Arrive on site. / Begin running pressure
T. Tom McCrory and Gary Crow. transducer cables
to the DRW-1 pump house and connecting
them to the Hermit 2000 data logger.
Hook up sequence is as follows:

Well No.	Data logger No.
DRW-1	Port 1
6GW1DB	Port 2
6GW1DA	Port 3
6GW1D	Port 4
6GW1S	Port 5
DP2	Port 6
DP1	Port 7
SP2	Port 8
SRW1	Port 9
6GW15D	Port 10

1400 Begin inputting set up parameters
(ID, linearity, scale, offset) for
each transducer.

1445 Begin placing pressure transducers in
wells and taping transducer cables
to the well casings, in the sequence
DP-2, 6GW15D, SRW-1, SP-2, 6GW1DB,
6GW1DA, 6GW1D, 6GW1S, DP-2.

1540 Check data logger to make sure all
transducers are recording and all
reference elevations are zero. All are
reading except Port 5, which is 6GW1S.
Check connections, loose connection on
cable reel, retighten, test, Port 5 OK.

1550 Check each transducer for response
by raising the transducer in the well
a few feet and observing the reading
of ~~ht~~ TAP on the data loggers. All
transducers are giving a signal consistent

Thomas A McCrory 2/18/96

Proj. No. 16032 Camp Lejeune Aquifer Tests WBS 0293001 2/18/96

with being raised in the well a couple feet.

1615 Store shipping boxes and police site.

1630 Leave site.

1645 Return to Holiday Inn.

TAM

Left blank

2/18/96

Thomas A McCrory 2/18/96

Proj. No. 16032 Camp Lejeune Aquifer Test WBS 0293001 2/19/96

0635 Leave Holiday Inn

0650 arrive on site

0700 H&S meeting

0715 Set up for step drawdown test

0930 Turn pump on. Present - Tom McCrory, Gary Crowe, Randy Smith, Curt Dockdn, Kai Mak, Dan Fisher (Baker), Jasper. Initial flow rate of 50 gpm draws water to within 6 feet of pump inlet in just a couple minutes, cavitation noises are heard. Cut back flow to 29 gpm; ~~flow stabilizes~~ THEN water level stabilizes at 4.7-4.8 feet above transducer, which is @ 1/2 feet above pump inlet.

0945 Shut down pump & allow well to recover.

Totalizer before pumping to establish flow rate 200 gallons.

After pumping to establish flow rate 1495 ~~1575~~ gallons

1000 Begin step drawdown

1130 After 90 minutes of pumping at 30 gpm, drawdown has stabilized at @ 69 feet below static (Δ < 0.005 feet over the last 40 minutes). We will use this rate for the sustained yield test to start tomorrow.

1200 Leave for lunch

1300 Return to site. Place duct tape over all outdoor observation wells to prevent any precipitation from entering them during the aquifer test. Down load data from Hermit and Troll into laptop computer, save data, print and review. Purchase raingauge and install it.

Thomas A McCrory 2/19/96

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outside well house for DRW-1.

1720 Leave site

1740 Arrive Holiday Inn.

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2/19/96

Thomas A McGrory 2/19/96

Proj No. 16032 Camp Lejeune Aquifer Test WBS 0293001 2/20/96

0635 Leave Holiday Inn

0650 Arrive at the site

0700 Health & Safety meeting.

0720 Set up Troll.

0734 Start Troll.

0740 Arrive DRW-1. Light rain is beginning to fall. No measurable precipitation (ppt) collected in rain gauge yet. Check water level in DP-2 = 16.65' below Top of Casing (TOC). Begin zeroing all reference points on data logger. Totalizer reads 4,473. Set up data logger for test.

0750 Start data logger; after 10 second delay for static readings, start pump. Initial flow rate is 50-56 gpm.

0755 After 5 minutes, flow rate is 29-30 gpm. Drawdown pattern is very similar to yesterdays, except water surface is about 1.0-1.5 feet lower than at the same point in the test yesterday.

0800 After 10 minutes, flow rate is 29 gpm. Rain is now heavier.

0810 After 20 minutes, flow rate is 28 gpm. Water level decline is slowing, less than 0.2 feet in the last 5 minutes.

0830 After 40 minutes, flow rate is 29 gpm. Barometric pressure at 0800 was 30.14 inches Hg.

0850 After 1 hour, flow rate is 29 gpm. Water level in DP-2 is 16.71 as measured by water level indicator. Change as indicated by pressure transducer is .03. Water level in DRW-1 appears to have stabilized at approximately 70' below static level.

Thomas A McCrory 2/20/96

Project No. 16032 Camp Lejeune Aquifer Test WBS0293001 2/20/96

Precipitation during the first hour of the test totaled approximately 0.1 inches. Rain is stopping and the sun has just come out.

0920 After 90 minutes, flow rate is 27 gpm. Water level is approximately 69.7' below static in DRW-1. A plot of the data from 0 to 60 minutes shows a definite response in 6GW1D and possible responses in DP-1 and DP-2.

0950 After 120 minutes, flow rate is 27 gpm. Water level indicator reads 16.73 in DP-2. Water level in DRW-1 is 69.762. No measurable ppt has fallen in the last hour.

1105 After approx 3 hrs (180 min) flow rate is 27 gpm

1115 Checked water level in DP2 : 16.83 bTOC original water level was 16.65 Δ = .18 datalogger reading Δ = .146. Beginning to rain fairly hard.

1130 Stop raining

1150 Barometric pressure is 30.07. No measurable ppt since 0950. Water level in DRW-1 (-69.683 compared to static) has not changed in the last 30 minutes; has risen 0.05 feet in the last 90 minutes.

1220 A fast moving low level cloud front has just moved across the site and a steady rain has begun to fall with intermittent gusts of wind. Pumping rate is 26 gpm.

1250 Water level meter indicates depth to water in DP-2 is 16.87. Flowrate is 25 gpm. Water level in DRW-1 is -69.683 (compared to static). No change in the last 90 minutes. Still raining steadily.

1350 Water level meter indicates depth to water in DP-2 is 16.91. Flow rate is 25 gpm.

Thomas A McCroey 2/20/96

Project No. 16032 Camp Lejeune Aquifer Test WBS 0293001 2/20/96

Ppt in the last 60 minutes was 0.25 inches; 0.35 inches cumulative since test began. Depth to water (compared to static) in DRW-1 is 69.683 feet; no change in the last 150 minutes. Still raining steadily.

1450 Water level indicator measures depth to water in DP-2 as 16.94'. Ppt in the last 60 minutes was 0.05 inches; 0.40 cumulative since test began. Rain has almost stopped. Pumping flow rate is 25 gpm. Depth to water in DRW-1 (compared to static) is -69.652. Water level has risen 0.03 feet in the last 60 minutes.

1550 Water level indicator measures depth to water in DP-2 as 16.97'. Ppt in the last 60 minutes was 0.05 inches; 0.45 cumulative since test began. Not raining at the moment. Depth to water in DRW-1 (compared to static) is -69.652; no change in the last 60 minutes. Pumping flow rate is 24 gpm.

1650 Water level indicator measures depth to water in DP-2 as 17.02'. Ppt in the last 60 minutes was 0.05 inches; 0.50 cumulative since ~~test~~ began. Drizzling at the moment. Depth to water in DRW-1 (compared to static) is 69.636; water has risen 0.016' in the last 60 minutes. Pumping flow rate is 24 gpm

1750 Water level indicator measures depth to water in DP-2 as 17.05'. No measurable ppt in the last 60 minutes. Not raining at present. Barometric pressure is 30.42 inches Hg. Pumping flow rate is 25 gpm. Depth to water in DRW-1 (compared to static) is 69.620; water has risen 0.016' in the last 60 minutes.

Thomas H McCrory 2/20/96

Proj. No. 16032 Camp Lejeune Aquifer Test WBS0293001 2/20/96

1850 Flashlight has not yet been delivered so cannot manually gauge DP-2. No measurable ppt has fallen in the last hour. Flow rate is 24 gpm. Depth to water in DRW-1 (compared to static) is 69.605; water has risen 0.015 feet in the last hour.

1950 No measurable ppt has fallen in the last hour. Flow rate is 25 gpm. Depth to water in DRW-1 (compared to static) is ~~25g~~ TAM 69.573; water has risen 0.032 feet in the last hour.

2050 No measurable ppt has fallen in the last hour. Flow rate is 24 gpm. Depth to water in DRW-1 (compared to static) is 69.542; water has risen 0.029 feet in the last hour.

2150 No measurable ppt has fallen in the last hour. Flow rate is 24 gpm. Depth to water in DRW-1 (compared to static) is 69.495; water has risen 0.047 feet in the last hour.

2250 No measurable ppt has fallen in the last hour. The sky appears to be clearing and stars are visible overhead. Flow rate is 25 gpm. Depth to water in DRW-1 (compared to static) is 69.479; water has risen 0.016 feet in the last hour.

2350 Flow rate is 25 gpm. Depth to water in DRW-1 (compared to static) is 69.463; water has risen 0.016 feet in the last hour.

2400 Gary Crook arrives for 12 hour shift.

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2/20/96

Thomas A McEnroe 2/20/96

Project No. 16032 Camp Lejeune Aquifer Test WBS0293001 2/21/96

115 Flowrate: 27 gpm
200 Flowrate: 24 gpm TOT 32120 gallons
325 Flowrate: 26 gpm TOT 34375 gallons
535 Flowrate: 27 gpm TOT 37590
755 Lance Laughmiller, an engineer w/ Lant Div on site. Reviewed the 24 hrs worth of data w/ him.
8:15 Lance Laughmiller off site to check out dig activities
8:20 Flow rate 27 gpm TOT 41890
8:45 Checked level in PPZ - 17.42 BTOC w/ water level indicator. Original level was 16.65 BTOC
Δ = .77 data logger is showing Δ = .740
8:55 Drawdown in DRW-1 is - 69.165 appears to have risen approx 1/2' since 7 hrs into test. May need to adjust flow rate. All other observation wells appear to increase their drawdown.
9:00 Will adjust flow a little: flow rate at present is 25.5 gpm Adjusted flow rate to 26 gpm
9:15 Adjusted flowrate to approx 27.5 gpm will monitor drawdown. At present -69.165
1000 will adjust flowrate some more. Present flow rate is 26 gpm, drawdown: -69.086
1030 Adjusted flow rate to 27.5 gpm drawdown -69.055'
1130 Flowrate: 26.5 gpm TOT: 46900 gallons
1150 Tom McCrory arrives for 12 hour shift. Data logger on 4 hour schedule. Review events of the last 12 hours with Gary. I note that in-line-flow meter on DRW-1, which was registering about 30-32 psi on the evening of the 20th, is now showing 16-18 psi, and the red handle with a square weight

Thomas A McCrory 2/21/96

Project No. 16032 Camp Lejeune Aquifer Test WBS 0293001 2/21/96

on the valve just "downstream" from the pressure gauge is ossilating up and down half an inch or so every second or two. The weather is sunny.

1315 Barometric pressure 30.04 inches of Hg. Measured distance from 6GW D1A to DRW-1 = 43 feet (by tape).

1400 Pumping flow rate = 25.5 gpm.

1550 Water level indicator measures depth to water in DP-2 as 17.49. This is a drawdown of 0.84' since the test started. The Hermit datalogger shows a drawdown of 0.848 for DP-2. Flow rate is 25 gpm. Drawdown in DRW-1 (relative to static) is -68.945 feet.

1800 A dense fog suddenly moved in and the temperature began to drop rapidly.

1950 Water level indicator measures depth to water in DP-2 as 17.55'. This is a drawdown of 0.90' since the test started. Flow rate is 23 gpm. Drawdown in DRW-1 (relative to static) is -68.662 feet.

2150 Drawdown in DRW-1 is -68.489. Flow rate is 23 gpm.

2350 Water level indicator measures depth to water in DP-2 as 17.57. This is a drawdown of 0.92' since the test started. Flow rate is 23 gpm. Drawdown in DRW-1 (relative to static) is 68.316. This is a change of +0.346' in 4 hours.

2400 Gary Crowe arrives.

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2/21/96

Thomas A McIroy 2/21/96

Project No: 16032 Camp Lejeune Aquifer Test W85029300l 2/22/96

140 Flowrate: 24 gpm 67490 gallons press: 15 psi drawdown 68.175' 3.736' of head above transducer. Will try to increase flow rate.

155 Flow rate: 25 gpm 67875 gallons press 15psi drawdown 68.159 3.752' of head above transducer: will monitor at this rate for awhile. Drawdown in observation wells appear to be holding or increasing slightly.

405 Flowrate: 25 gpm 71110 gallons press 15 psi

500 Checked DPZ-17.63 original level 16.65 Δ = .98' data logger showing drawdown of .931 B. static water level

730 Flowrate 25.5 gpm 76180 gallons press 15 psi drawdown in DRW-1: 67.672' below static Head above transducer ~~4.272~~ 4.25' ~~below~~

930 Checked water level in DPZ ~~as~~ 17.64 original level was 16.65 Δ = .99' data logger showing a drawdown of .963. Flow rate is: 25 gpm 16 psi TOT: 79460. Drawdown is 67.483' Head 4.427' above Transducer

1155 Flow rate: 26 gpm TOT 82990 15 psi

1200 Tom McCrory arrives; Gary Crowe leaves

1400 Flow rate is 26 gpm

1550 Water level indicator measures depth to water in DP-2 as 17.65', a drawdown of 1.00' since the test began. The data logger indicates a drawdown of 1.001', very good agreement. The flow rate is 24 gpm. The drawdown in DRW-1 (relative to static) is 67.059. This is a rise of about 0.6 feet since 0730 today.

1800 Flow rate is 27 gpm. Approximately 90,000 gallons have been pumped since the test began.

Thomas A McCrory 2/22/96

Project No. 16032 Camp Lejeune Aquifer Test WBS0293001 2/22/96

1950 Water level indicator measures depth to water in DP-2 as 17.67, a drawdown of 1.02' since the test began. The data logger indicates a drawdown of 1.020. The pumping rate is 26 gpm.

2350 Water level in DP-2 is 17.70 (by water level indicator), a drawdown of 1.05' since the test began. The data logger indicates a drawdown of 1.033'. The flow rate is 26 gpm. The drawdown in DRW#1 is 66.918', a rise of 0.015' in the last 4 hours.

2/23/96 0000 - G. Crowe onsite T. McCrory offsite

100 Pumping rate: 26 gpm pressure is 15psi 66.902' is drawdown in DRW-1

310 Checked 6GW38D - max drawdown achieved .048 after 50 min then water level rose and at present is above the original water level.

630 Flowrate 26 gpm. Drawdown in DRW1: 66.918' PSI = 15. Checked water level in DP2: 17.79 Δ = 1.14'. Data logger shows 1.055'

753 Stop pumping TOT 114657 gallons. Begin recovery test.

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2/23/96

Thomas A McCrory 2/23/96

Project No. 16032 Camp Lejeune Aquifer Test WBS 0293001 2/24/96

0800 Arrive on site. Bady Crowe begins collection, recording, tabularizing and printing data. Tom McCrory begins supervision of taking up pressure transducers and laying out network for shallow aquifer test.

1400 Hook up sequence is as follows: M

Well No.	Data logger No.
SRW-1	Port 1
SP2	Port 2
SP1	Port 3
6GW34	Port 4
SP3	Port 5
DRW-1	Port 6
6GW15	Port 7
6GW33	Port 8

Begin inputting set up ~~linearity~~ TAM parameters (ID, linearity, scale, offset) for each transducer.

Original setting on flowmeter is 000 gallons

1500 Start pump with line opened to 45°. Flow rate (initial) is @ 3 gpm.

1515 Flow rate is 2.6 gpm

1540 Flow rate is 2.4 gpm. Water level in SRW-1 has almost leveled off at about 3.6' below static. We are increasing the line opening to 60° (compared to off).

1550 Due to lack of response in water level, we have fully opened the line. However flow rate has only increased to 2.8 gpm. As the pump is rated to 7 gpm, this suggests that something is preventing free flow of the water. To check the pump and pipe to the surface, we stop the

Thomas A McCrory 2/24/96

test, pull the pump out of the well, and place it in a 55 gallon drum full of water. When running it in this mode, the flow rate is about 6 gpm and no leaks are visible in the hose. This suggests that pinching in the fire hose connecting the pump to the frac tank may be what is causing the reduced flow.

1800 Leave site.

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2/24/96

Thomas A McCrory 2/24/96

Project No. 16032 Camp Lejeune Aquifer Test 085029300l 2/25/96

0800 Gary and I arrive on site. Discuss situation with Alan. He suggests purchasing 1" ID PVC (Sch 10) pipe to install from the pump house to the frac tank.

1030 The pipe is installed. We begin putting the pump back in the well, reconnecting all fittings, and restablishing the electrical connections.

1230 Turn on pump in SRW-1 with line open all the way. The instantaneous flow is about 6 gpm. After half an hour, the flow rate is 5-5.5 gpm.

1325 The drawdown in SRW1 is 9.5 feet, and the rate of drawdown has gone from 0.07 feet per minute to 0.03 feet per minute of the last 20 minutes. As the transducer is approximately 16 feet below static water this appears to be a sustainable drawdown.

1330 Turn off pump. We will allow 2 hours for the aquifer to completely recover and then begin the 72 hour test.

1530 Set all reference elevations of transducers to zero. Start data logger. After ~~30 seconds~~ 1 minute TAM start pumps.

1535 Flow rate is approximately 6gpm. Drawdown is 5.6 feet in SRW-1. Drawdown is 0.634 in SP-7. Depth to water by water level indicator is 17.98' in SP7.

1540 Flow rate is 5gpm. Drawdown is

Thomas A McCrory 2/25/95

Project No 16032 Camp Lejeune Aquifer Test WBS 0293001 2/25/96

8.958 feet in SRW-1.

1545 ~~Drawdown in SRW-1 is~~ TAM SRW-1 is 10.441 feet. Drawdown rate of increase id about 0.2 feet per minute.

1550 Flow rate is approximattoly 5.5 gpm. Drawdown is 11.410 feet.

1553	Drawdown	11.622	
1554	"	11.732	Δ = 0.10
1555	"	11.893	Δ = 0.16
1556	"	12.039	Δ = 0.14
1557	"	12.159	Δ = 0.12
1558	"	12.280	Δ = 0.12
1559	"	12.380	Δ = 0.10
~~1560~~ TAM 1600	"	12.451	Δ = 0.07
1601	"	12.551	Δ = 0.10
1602	"	12.631	Δ = 0.08
1603	"	12.737	Δ = 0.11
1604	"	12.848	Δ = 0.11
1605	"	12.913	Δ = 0.075
1606	"	12.998	Δ = 0.085
1607	"	13.043	Δ = 0.045
1608	"	13.124	Δ = 0.081
1609	"	13.184	Δ = 0.060
1610	"	13.224	Δ = 0.040
1611	"	13.275	Δ = 0.051
1612	"	13.315	Δ = 0.040
1613	"	13.365	Δ = 0.050
1614	"	13.405	Δ = 0.040
1614	"	13.451	Δ = 0.046
1615	"	13.496	Δ = 0.044
1616	"	13.536	Δ = 0.040
1617	"	13.566	Δ = 0.030
1618	"	13.616	Δ = 0.050
1619	"	13.667	Δ = 0.051
1620	"	13.697	Δ = 0.030

Thomas A McCrory 2/25/96

Project Ab16032 Camp Lejeune Aquifer Test WBS0293001 2/25/96

1621 Flow rate is about 4.6 gpm

	Drawdown	13.737	Δ = 0.040
1622	"	13.767	Δ = 0.030
1624	"	13.777	Δ = 0.010
1625	"	13.812	Δ = 0.035
1626	"	13.837	Δ = 0.025
1627	"	13.868	Δ = 0.031
1628	"	13.878	Δ = 0.010
1629	"	13.898	Δ = 0.020
1630	"	13.923	Δ = 0.025
1631	"	13.948	Δ = 0.025
1632	"	13.968	Δ = 0.020

Flow rate is 5 gpm

1633	Drawdown	13.988	Δ = 0.020
1634	"	14.003	Δ = 0.015
1635	"	14.028	Δ = 0.025

The drawdown appears to be stabilizing, though at a level about 4.5 feet lower than in the step drawdown test performed earlier today. I will check it again shortly. Flow into frac tank is proceeding smoothly

1640 Drawdown 14.149 Δ = 0.121/5 min = Δ 0.024/min.

Atmospheric pressure = 30.08

1650 Drawdown 14.345 Δ = 0.196/5 min = Δ 0.039/min

1700 Flow rate = 5.2 gpm

Drawdown 14.656 Δ = 0.311 /5 min = Δ = 0.06/min

1710 Drawdown = 14.898 Δ = 0.242 / 5 min = Δ = 0.048/min.

1720 Drawdown = 15.053 Δ = 0.155/5 min = Δ = 0.031 /min.

Flow rate = 4.8 gpm

Thomas A McCoy 2/25/96

Project No 16032 Camp Lejeune Aquifer Test WBS 0293001 2/25/96

1730 Drawdown = 15.144 Δ = 0.091/5 min = 0.02/min.

Flowrate = 5 gpm

1740 Drawdown = 15.214 Δ = 0.070/5 min = 0.014/min.

1750 Drawdown = 15.290 Δ = 0.076/5 min = 0.015/min

Flowrate = 5 gpm

1800 Drawdown = 15.320 Δ = 0.030/5 min = 0.006/min.

1830 Drawdown = 15.686 Δ = 0.366/30 min = 0.012/min

Transducer is only 16 feet below static. As this does not leave a sufficient margin for error, I am terminating the test and will allow the well to recover for at least ~~[illegible]~~ hours before attempting to determine the sustainable yield.

1130 While reviewing the recovery in the wells, I've noticed that the depth below the surface ~~show~~ TAM of the transducer is 36 feet. As the well is only 35 feet deep and the bottom few feet are filled with the pump, this is suspicious. When I lift the transducer approximately 2 feet in its drop pipe the data logger records a change of almost 4 feet. Upon checking, I've discovered that the linearity, scale and offset for SRW-1 have been erased. I've reinput them. This seems to explain why the drawdown ~~is too~~ TAM appeared so

Thomas A McEwen 2/25/96

much greater during the continuous test than during the step drawdown test. I will reprogram the data logger to begin a new test and start it at 2130.

~~2110~~ 1210 TAM Preliminary reset all references to zero.

2130 Start 72 hour sustained yield test in SRW-1. Pump and instruments appear to be running correctly. Initial flow rate is 5.5 gpm.

2145 Flow rate is 4.5 gpm. Drawdown in SRW-1 is 6.74 feet, much more like that in the step drawdown test performed at 1230 today.

2200 Flow rate is 5.5 gpm. Drawdown in SRW-1 is 8.11 feet. The rate of drawdown is declining, and there are still almost 9 feet of head above the transducer. Barometric pressure is 30.08" Hg.

2215 Flow rate is 5 gpm. Drawdown in SRW-1 is 8.63 feet

2230 Flow rate is 5 gpm. Drawdown in SRW-1 is 8.95 feet.

2245 Flow rate is 4.6 gpm. Drawdown in SRW-1 is ~~TAM~~ 9.17 feet.

2300 Flow rate is 5 gpm. Drawdown in SRW-1 is 9.26 feet.

2315 Flow rate is 5 gpm. Drawdown in SRW-1 is 9.35 feet

2330 Flow rate is 4.8 gpm. Drawdown in SRW-1 is 9.41 feet.

2345 Flow rate is 5 gpm. Drawdown in SRW-1 is 9.49 feet.

Thomas A McCrory 2/25/96

Project No. 16032 Camp Lejeune Aquifer Test WB5029J001 2/26/96

1000 Flow rate is 5 gpm. Drawdown in SRW-1 is 9.539 feet. Gary has arrived to take the midnight to noon shift.

Gr 130 ~~1330~~ Flowrate is 5 gpm drawdown in SRW1 - 9.700'

400 Flowrate is 4.5 gpm drawdown in SRW1 - 9.885'

900 Flowrate is approx 5 gpm drawdown in SRW1 - 9.904 Head is 7.267

955 Flowrate is approx 4.5 gpm drawdown in SRW-1 10.040 head: 7.131 above transducer

1200 Barometric pressure is 29.975. Depth to water in SRW-1 is 10.10. Flow rate is 5 gpm.

1300 Drawdown seems to be stabilizing at approximately 10.1 feet below static. Flow rate is @ 4.5 gpm.

1500 Drawdown in SRW-1 is 10.071 feet. Flow rate is 4.5 gpm.

1600 Drawdown in SRW-1 is 10.134 feet. Flow rate is 5 gpm. Weather has been clear since the start of the test.

1730 Flow rate is 4.8 gpm. Drawdown in SRW-1 is 10.216 feet

1930 Flow rate is 5 gpm. Drawdown in SRW-1 is 10.333.

2030 Flow rate is 4.5 gpm. Drawdown in SRW-1 is 10.386

2130 Drawdown in SRW-1 is 10.421 feet. Over the last 6 hours, drawdown has occured at 0.03-0.04 feet per hour. So long as the rate of drawdown does not increase, there is plenty of head (currently 6.76 feet) for the remainder of the test (48 hours). Flow rate is 4.8 gpm.

Thomas A McEvoy 2/26/96

Project No 16032 Camp Lejeune Aquifer Test WBS0293001 2/26/96

2330 Drawdown in SRW-1 is 10.456, a change of 0.035 feet in the last hour. Flow rate is about 4.5 gpm. Barometric pressure is 29.91" Hg.

2330 Drawdown in SRW-1 is 10.449. Flow rate is 5 gpm.

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2/26/96

Thomas A McGowan 2/26/96

Project No. 16032 Camp Lejeune Aquifer Test WBS 0293001 2/27/96

0010 Gary arrives.

0135 Approx 5 gpm SRW1 drawdown is 10.468 head 6.703' above transducer.

0345 Approx 5 gpm SRW1 drawdown is 10.493 head 6.677' above transducer

625 Approx 5 gpm flowrate SRW1 drawdown is 10.553 head 6.618' above transducer.

6:45 tried to call base weather service. Line was busy will call again later to get barometric data.

8:05 Barometric pressure : 29.96" Hg compared to 29.91 @ 2230 hrs.

1025 Flowrate : 5 gpm SRW1 drawdown is 10.720 head 6.451' above transducer

1200 Barometric pressure : 29.99" Hg. Drawdown in SRW-1 is 10.714'. Flow rate is 4.8 gpm. Tom McCrory has arrived.

1330 Drawdown in SRW-1 is 10.654. Flow rate is 5.2 gpm.

1500 Drawdown in SRW-1 is 10.635. Flow rate is 5 gpm. Checked on the hook up of the transducers to determine if 6GW15 is hooked up to Port 7 on the data logger. It is.

1730 Drawdown in SRW-1 is 10.682. Flow rate is 4.6 gpm. Walked line to frac tank - no leaks.

1930 Drawdown in SRW-1 is 10.695. Flow rate is 5.2 gpm.

2130 Drawdown in SRW-1 is 10.730. Flow rate is 5 gpm. Barometric pressure is 29.85.

2330 Drawdown in SRW-1 is 10.714. Flow rate is 4.8 gpm, Gary has just arrived.

Thomas A McCrory 2/27/96

Project No. 16032 Camp LeJeune Pumping Test
WBS 0293001 2/28/96

~~0000 Gary Crowe onsite.~~ Gwe

0200 Flowrate: 5 gpm drawdown in SRW1 is 9.454 head is 7.717' above transducer. TOT. 16358. Clear sky, no rain.

0400 Flowrate is approx 4 gpm drawdown in SRW1 is 9.406' and head 7.764 feet above transducer. TOT. 16900.

0415 Barometric pressure as measured by base weather station is 29.73" Hg

0600 Flowrate is approx 5 gpm drawdown in SRW1 is 9.457' Head is 7.714' above transducer

0610 TOT. 17480 gallons.

0830 Approx 5 gpm drawdown is 9.533 in SRW1 head is 7.638" above transducer TOT 18168 gallons

0900 Barometric pressure: 29.75" Hg

1115 Tom McCrory arrives.

00 Drawdown is 9.523 ~~in~~ SRW-1. Flowrate is 4.6 gpm. The weather is changing. Skies are now overcast, and the wind is starting to pick up.

1300 The rain gauge is empty. An intermittent light rain has just begun.

1330 The rain has stopped for the time being. Drawdown in SRW-1 is 9.596. Flow rate is 4.6 gpm. Total gallons pumped since the start of the test = 19,556, for an average pumping rate of 5.09 gpm.

OVER ESTIMATED (false starts not counted)

Corrected Estimate
$\frac{19,556 - 16,358}{11.5 \text{ hours}}$ = 4.6 gpm

1340 The rain has started again, heavier this time.

1530 The rain has stopped. Total accumulation is less than 1/10 of an inch. Drawdown in SRW-1 is 9.567

Thomas A McCrory 2/28/96

D57 6-25-96

Project No. 16032 Camp Lejeune Aquifer Test WBS0293001 2/28/96,

Flow rate is 4.8 gpm. Barometric pressure is 29.79" Hg.

1535 ~~1335~~ TAM While backing up PCT truck alongside the frac tank, the PVC line going up to the frac tank is knocked down. This is observed and corrected within a couple minutes. Upon returning to SRW-1, I notice that the water level has dropped to 9.9' below static. Over the next 10 minutes, it returns to 9.7' below static.

1730 Drawdown in SRW-1 is 9.558'. Flow rate is 4.6 gpm. It has not rained since approximately 1530.

2100 Barometric pressure is 29.77" Hg.

2130 Flow rate is 4.6 gpm. Drawdown in SRW-1 is 9.630. Dumped all data into laptop computer; Checked LOTUS to confirm \lejeune\72HRSHAL\SHA.PRN.

2140 Start recovery test. Reset data logger to new step, turn off pump, shut valve on PVC line to frac tank. Datalogger shows rapid recovery in SRW-1 (6.49 feet of drawdown after 2.5 minutes). Will leave data logger to operate overnight; check results in the morning.

2200 Leave site.

TAM

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2/28/96

Thomas A McCrory 2/28/96

Project No. 16032 Camp Lejeune Aquifer Test WBS0293001 2/29/96

0800 G. Crowe + T. McCrary arrive onsite. Begin down loading file from data logger.
0900 T. McCrary makes arrangements to fly out today & will leave on 1300 flight back to Atlanta. I will continue to observe recovery and download info and prepare 72 hr pumping test data.
1700 Off site.

3/01/96.
700 G Crowe arrives onsite. Downloads information from data logger. Begins creating computer files for Baker Engineering and a copy for OHM Corp to be left on site.
0800 Direct S. Pyle and Jim Lesert to pull probes and decon them.
01130 Finished deconning probes and packing them will remove probes from wells after lunch. Will leave field book in computer case to be carried by M. Haugen to Norcross office.

Gary W. Crowe.

Attachment C –Recorded Data during the Shallow Continuous-Rate Pumping Test

CAMP LEJEUNE AQUIFER TEST
PROJECT NUMBER 16032 WBS 0293001
SRW-1 (SHALLOW WELL) 72 HOUR PUMPING TEST
FEBRUARY 25-28, 1996
FLOW RATE: 5 GPM
DRAWDOWN IN FEET

ELAPSED TIME IN MINUTES	SRW-1	SP2	SP1	6GW34	SP3	DRW-1	6GW1S	6GW33
0.00	0.00	0.00	-0.01	-0.01	-0.00	-0.02	-0.01	-0.00
0.50	1.06	0.03	-0.01	-0.01	0.00	-0.02	-0.01	-0.00
1.00	1.58	0.10	0.01	-0.01	-0.00	-0.02	-0.01	-0.01
1.50	2.04	0.17	0.03	-0.01	-0.00	-0.02	-0.02	-0.01
2.00	2.53	0.22	0.04	-0.01	-0.01	-0.02	-0.01	-0.01
2.50	2.86	0.28	0.07	-0.01	-0.01	-0.02	-0.01	-0.01
3.00	3.15	0.33	0.09	-0.01	-0.00	-0.02	-0.01	-0.01
3.50	3.41	0.38	0.10	0.01	0.00	-0.02	-0.01	-0.01
4.00	3.68	0.42	0.13	0.01	-0.00	0.00	-0.01	-0.01
4.50	3.93	0.46	0.14	0.01	0.00	-0.02	-0.01	-0.01
5.00	4.28	0.51	0.16	0.01	0.00	-0.02	-0.01	-0.01
5.50	4.43	0.55	0.18	0.01	-0.00	-0.02	-0.01	-0.00
6.00	4.59	0.59	0.19	0.01	0.00	-0.02	-0.01	-0.00
6.50	4.74	0.62	0.21	0.02	0.00	-0.02	-0.02	-0.01
7.00	4.90	0.65	0.21	0.01	0.00	-0.02	-0.02	-0.01
7.50	5.03	0.68	0.23	0.03	0.01	-0.02	-0.02	-0.01
8.00	5.17	0.70	0.24	0.03	0.01	-0.02	-0.02	-0.01
8.50	5.40	0.73	0.25	0.02	0.01	-0.02	-0.02	-0.01
9.00	5.53	0.75	0.27	0.03	0.01	-0.02	-0.02	-0.01
9.50	5.64	0.77	0.28	0.03	0.01	-0.02	-0.02	-0.00
10.00	5.76	0.79	0.30	0.03	0.01	-0.02	-0.02	-0.01
11.00	6.10	0.82	0.32	0.05	0.01	0.00	-0.01	-0.00
12.00	6.28	0.85	0.34	0.06	0.01	-0.02	-0.01	-0.01
13.00	6.44	0.90	0.36	0.06	0.02	-0.02	-0.01	-0.01
14.00	6.59	0.94	0.38	0.06	0.02	0.00	-0.02	-0.01
15.00	6.74	0.97	0.39	0.06	0.02	0.00	-0.01	-0.01
20.00	7.33	1.09	0.46	0.09	0.03	0.00	-0.01	-0.01
25.00	7.76	1.16	0.51	0.11	0.04	0.00	-0.01	-0.01
30.00	8.11	1.22	0.56	0.13	0.05	0.00	-0.02	-0.01
35.00	8.31	1.26	0.58	0.15	0.06	-0.02	-0.03	-0.01
40.00	8.48	1.29	0.62	0.16	0.06	-0.02	-0.02	-0.01
45.00	8.63	1.32	0.64	0.18	0.07	0.00	-0.01	-0.01
50.00	8.76	1.35	0.66	0.19	0.07	0.00	-0.01	-0.01
55.00	8.86	1.37	0.68	0.20	0.08	-0.02	-0.02	-0.01
60.00	8.95	1.38	0.69	0.21	0.08	-0.02	-0.02	-0.01
90.00	9.26	1.46	0.76	0.25	0.10	0.00	-0.02	-0.01
120.00	9.41	1.51	0.80	0.28	0.12	0.00	-0.02	-0.01
150.00	9.53	1.55	0.84	0.31	0.13	-0.02	-0.02	-0.01
180.00	9.60	1.57	0.87	0.33	0.14	-0.02	-0.02	-0.01
210.00	9.66	1.59	0.89	0.35	0.16	0.00	-0.00	-0.01
240.00	9.72	1.61	0.91	0.37	0.17	0.00	-0.01	-0.01
270.00	9.76	1.62	0.92	0.37	0.17	0.00	-0.00	-0.01
300.00	9.82	1.63	0.93	0.38	0.18	0.00	-0.01	-0.01
360.00	9.90	1.65	0.95	0.39	0.19	-0.02	-0.02	-0.01
420.00	9.81	1.67	0.97	0.42	0.20	-0.02	-0.01	-0.01
480.00	9.88	1.69	0.99	0.43	0.22	0.00	0.01	0.00
540.00	9.92	1.70	1.00	0.44	0.23	0.00	0.00	0.01
600.00	9.96	1.72	1.02	0.45	0.25	0.00	0.01	0.01
660.00	10.00	1.73	1.02	0.46	0.25	0.00	0.01	0.02
720.00	10.03	1.73	1.03	0.47	0.25	0.02	0.05	0.04
780.00	10.05	1.74	1.03	0.46	0.26	0.00	-0.00	-0.01
840.00	10.10	1.74	1.04	0.47	0.27	0.03	0.07	0.06
900.00	10.11	1.74	1.04	0.46	0.25	0.02	-0.02	0.00
960.00	10.08	1.74	1.02	0.45	0.25	0.03	0.00	-0.01
1020.00	10.07	1.73	1.02	0.45	0.25	0.03	-0.04	-0.03
1080.00	10.11	1.74	1.02	0.45	0.25	0.03	-0.04	-0.04
1140.00	10.17	1.75	1.04	0.47	0.26	0.02	-0.01	-0.03
1200.00	10.22	1.77	1.05	0.49	0.28	0.03	-0.01	-0.03
1260.00	10.28	1.78	1.07	0.50	0.29	0.03	0.00	-0.01
1320.00	10.33	1.81	1.09	0.52	0.32	0.03	0.02	0.01
1380.00	10.39	1.81	1.10	0.53	0.32	0.05	0.02	0.02
1440.00	10.42	1.82	1.11	0.54	0.33	0.05	0.02	0.02
1680.00	10.47	1.83	1.12	0.54	0.33	0.06	0.01	0.02
1920.00	10.56	1.87	1.15	0.58	0.37	0.09	0.03	0.03
2160.00	10.67	1.92	1.20	0.62	0.42	0.11	0.08	0.10
2400.00	10.65	1.88	1.16	0.58	0.39	0.13	0.03	0.04
2640.00	10.68	1.90	1.17	0.60	0.40	0.13	0.02	0.03
2880.00	10.73	1.91	1.19	0.61	0.41	0.11	0.02	0.03
3120.00	9.50	1.83	1.15	0.59	0.39	0.11	0.00	0.02
3360.00	9.45	1.83	1.15	0.60	0.39	0.11	0.01	0.03
3600.00	9.52	1.85	1.17	0.61	0.40	0.13	0.07	0.08
3840.00	9.60	1.88	1.19	0.64	0.44	0.14	0.03	0.03
4080.00	9.56	1.88	1.20	0.65	0.44	0.14	0.04	0.05
4320.00	9.63	1.92	1.23	0.68	0.48	0.17	0.05	0.07

CAMP LEJEUNE AQUIFER TEST
PROJECT NUMBER 16032 WBS 0293001
SRW-1 (SHALLOW WELL) 24 HOUR RECOVERY TEST FOLLOWING 72 HOUR PUMPING TE:
FEBRUARY 28-29, 1996
DRAWDOWN IN FEET

ELAPSED TIME IN MINUTES	SRW-1	SP2	SP1	6GW34	SP3	DRW-1	6GW1S	6GW33
0	9.655	1.931	1.236	0.695	0.492	0.157	0.07	0.088
0.5	9.047	1.925	1.242	0.695	0.488	0.157	0.073	0.088
1	8.395	1.912	1.242	0.688	0.495	0.157	0.073	0.088
1.5	7.72	1.893	1.23	0.695	0.492	0.157	0.073	0.091
2	7.089	1.865	1.23	0.695	0.492	0.157	0.073	0.085
2.5	6.493	1.827	1.223	0.701	0.492	0.157	0.073	0.094
3	5.96	1.786	1.198	0.688	0.501	0.173	0.073	0.088
3.5	5.43	1.735	1.186	0.695	0.498	0.157	0.076	0.097
4	4.963	1.681	1.167	0.695	0.495	0.157	0.079	0.094
4.5	4.622	1.637	1.148	0.688	0.495	0.157	0.079	0.094
5	4.319	1.595	1.129	0.695	0.488	0.157	0.079	0.094
5.5	3.902	1.545	1.117	0.682	0.495	0.173	0.082	0.094
6	3.624	1.5	1.092	0.688	0.488	0.173	0.082	0.088
6.5	3.397	1.456	1.079	0.688	0.492	0.173	0.082	0.091
7	3.195	1.415	1.06	0.682	0.485	0.173	0.079	0.091
7.5	3.024	1.383	1.048	0.676	0.488	0.173	0.082	0.094
8	2.86	1.351	1.041	0.676	0.488	0.173	0.089	0.094
8.5	2.699	1.316	1.016	0.669	0.485	0.189	0.086	0.091
9	2.569	1.291	1.004	0.663	0.482	0.173	0.082	0.097
9.5	2.449	1.265	0.991	0.65	0.479	0.157	0.082	0.085
10	2.297	1.243	0.979	0.663	0.476	0.173	0.086	0.094
11	2.057	1.199	0.96	0.65	0.482	0.173	0.086	0.094
12	1.89	1.154	0.935	0.644	0.479	0.173	0.086	0.094
13	1.748	1.12	0.916	0.644	0.479	0.173	0.082	0.094
14	1.628	1.088	0.897	0.644	0.479	0.173	0.086	0.1
15	1.517	1.059	0.884	0.638	0.476	0.189	0.092	0.1
20	1.157	0.936	0.815	0.606	0.46	0.173	0.089	0.1
25	0.973	0.85	0.759	0.587	0.447	0.173	0.086	0.094
30	0.856	0.79	0.721	0.562	0.434	0.173	0.086	0.094
35	0.777	0.742	0.684	0.543	0.425	0.189	0.079	0.088
40	0.739	0.71	0.659	0.524	0.415	0.173	0.076	0.091
45	0.708	0.685	0.633	0.511	0.412	0.173	0.076	0.091
50	0.679	0.653	0.608	0.492	0.396	0.173	0.07	0.085
55	0.651	0.628	0.583	0.473	0.387	0.173	0.057	0.085
60	0.629	0.602	0.558	0.455	0.377	0.173	0.047	0.078
90	0.572	0.542	0.502	0.423	0.371	0.173	0.07	0.081
120	0.483	0.472	0.439	0.379	0.349	0.173	0.06	0.075
150	0.426	0.428	0.395	0.347	0.33	0.173	0.057	0.075
180	0.385	0.399	0.376	0.334	0.33	0.189	0.066	0.081
210	0.344	0.364	0.338	0.309	0.311	0.189	0.06	0.078
240	0.306	0.33	0.307	0.284	0.288	0.189	0.047	0.072
270	0.287	0.314	0.288	0.278	0.285	0.189	0.054	0.075
300	0.275	0.307	0.288	0.278	0.288	0.189	0.07	0.085
360	0.252	0.288	0.276	0.271	0.282	0.189	0.089	0.091
420	0.249	0.288	0.276	0.278	0.295	0.189	0.105	0.11
480	0.249	0.285	0.276	0.284	0.301	0.205	0.121	0.125
540	0.24	0.276	0.269	0.278	0.298	0.221	0.114	0.129
600	0.237	0.276	0.263	0.278	0.298	0.221	0.124	0.141
660	0.227	0.266	0.257	0.271	0.295	0.236	0.153	0.16
720	0.211	0.247	0.238	0.246	0.279	0.236	0.086	0.129
780	0.196	0.234	0.225	0.227	0.269	0.236	0.051	0.122
840	0.177	0.215	0.207	0.221	0.257	0.252	0.133	0.144
900	0.154	0.193	0.182	0.195	0.234	0.236	0.063	0.103
960	0.135	0.174	0.169	0.183	0.219	0.252	0.102	0.119
1020	0.12	0.158	0.156	0.176	0.206	0.252	0.082	0.103
1080	0.113	0.155	0.15	0.17	0.203	0.252	0.098	0.11
1140	0.113	0.155	0.15	0.176	0.206	0.252	0.089	0.116
1200	0.129	0.168	0.163	0.195	0.212	0.252	0.108	0.125
1260	0.132	0.171	0.169	0.202	0.219	0.252	0.118	0.144
1320	0.126	0.165	0.163	0.195	0.215	0.268	0.111	0.129
1380	0.126	0.168	0.163	0.195	0.215	0.268	0.118	0.144
1440	0.11	0.145	0.144	0.176	0.196	0.268	0.095	0.132
1680	0.075	0.111	0.106	0.139	0.165	0.268	0.076	0.119
1920	0.066	0.104	0.106	0.145	0.155	0.252	0.092	0.119

Attachment D – Recorded Data during the Deep Continuous-Rate Pumping Test

CAMP LEJEUNE AQUIFER TEST
PROJECTI NUMBER 16032 WBS 0293001
DRW-1 (DEEP WELL) 72 HOUR PUMPING TEST
FEBRUARY 20-23, 1996
FLOW RATE: 27 GPM
DRAWDOWN IN FEET

ELAPSED TIME IN MINUTES	DRW-1	6GW1DB	6GW1DA	6GW1D	6GW1S	DP2	DP1	SP2	SRW-1	6GW15D
0	-0.015	0.009	0.012	0.012	0.003	0	0.012	0.006	0.003	0
0.5	15.97	0.009	0.012	0.012	0.003	0.006	0.012	0.006	0.003	0
1	33.646	0.009	0.006	0.019	0.003	0	0.012	0.009	0.003	0
1.5	46.608	0.009	0.012	0.038	0.003	0	0.012	0.009	0.003	0
2	56.183	0.009	0.012	0.07	0.003	0.006	0.012	0.009	0.003	0
2.5	61.81	0	0.006	0.121	0	0	0.019	0.009	0.003	0
3	64.451	0.009	0.006	0.178	0.003	0.006	0.012	0.009	0.003	0
3.5	66.242	0.009	0.006	0.242	0	0.006	0.019	0.012	0.003	0
4	67.201	0	0.006	0.312	0	0.006	0.019	0.012	0.006	0
4.5	67.499	0	0	0.382	0	0.006	0.019	0.009	0.006	0
5	67.75	0	0	0.446	-0.003	0.006	0.019	0.009	0.003	0
5.5	67.986	0	0	0.509	-0.003	0.006	0.019	0.012	0.006	0
6	68.19	-0.009	-0.006	0.554	-0.003	0.006	0.019	0.012	0.003	0
6.5	68.348	-0.009	-0.006	0.605	-0.003	0.006	0.019	0.009	0.003	0
7	68.52	-0.009	-0.012	0.649	-0.003	0.006	0.012	0.012	0.003	0
7.5	68.646	-0.009	-0.012	0.694	0	0	0.025	0.015	0.006	0
8	68.772	-0.009	-0.012	0.732	0	0.006	0.019	0.015	0.009	0
8.5	68.882	-0.009	-0.018	0.77	0.003	0.006	0.012	0.022	0.015	0
9	68.992	-0.009	-0.018	0.802	0.003	0.012	0.019	0.019	0.015	0
9.5	69.055	-0.019	-0.018	0.828	0	0.006	0.019	0.015	0.009	0
10	69.165	-0.019	-0.025	0.853	0.003	0.006	0.019	0.015	0.012	0
11	69.306	-0.019	-0.025	0.904	0	0.006	0.025	0.019	0.015	0
12	69.416	-0.019	-0.031	0.949	0.003	0.012	0.025	0.015	0.006	0
13	69.526	-0.028	-0.031	0.981	0	0.012	0.025	0.015	0.012	0
14	69.605	-0.028	-0.037	1.019	-0.003	0.006	0.025	0.006	0.006	0
15	69.652	-0.038	-0.037	1.044	-0.003	0.012	0.025	0.006	0.003	0
20	69.84	-0.047	-0.044	1.165	-0.006	0.006	0.025	0.006	0.003	0
25	69.919	-0.057	-0.044	1.248	-0.009	0.012	0.031	0.003	0	0
30	69.982	-0.066	-0.037	1.312	-0.006	0.019	0.038	0.003	-0.003	0
35	69.997	-0.076	-0.025	1.363	-0.003	0.019	0.044	0.003	0	0
40	70.06	-0.095	-0.018	1.395	-0.003	0.019	0.044	0	-0.006	0
45	70.076	-0.095	-0.006	1.427	-0.003	0.025	0.05	0.003	-0.003	0
50	70.076	-0.095	0.012	1.458	0	0.031	0.057	0.003	-0.003	0
55	70.076	-0.104	0.025	1.484	0.003	0.031	0.069	0.003	-0.003	0
60	70.029	-0.104	0.037	1.509	0.006	0.031	0.069	0	0	0
90	69.777	-0.085	0.132	1.605	0.012	0.07	0.107	0.009	0.003	0
120	69.762	-0.066	0.189	1.662	0.022	0.095	0.133	0.009	-0.003	0
150	69.746	-0.038	0.246	1.713	0.034	0.108	0.158	0.015	0.006	0
180	69.73	-0.009	0.29	1.739	0.037	0.134	0.177	0.015	0.006	0
210	69.699	0.028	0.335	1.764	0.037	0.153	0.19	0.015	0.009	0
240	69.683	0.066	0.366	1.79	0.025	0.172	0.215	0.009	0.003	0
270	69.683	0.085	0.385	1.802	0.041	0.191	0.222	0.006	-0.003	0
300	69.683	0.114	0.411	1.815	0.047	0.216	0.241	0.015	0.009	0
360	69.683	0.152	0.43	1.822	0.041	0.255	0.266	0.012	0.003	0
420	69.652	0.18	0.442	1.834	0.015	0.287	0.292	-0.012	-0.015	0.003
480	69.652	0.228	0.48	1.86	0.047	0.325	0.311	0.006	0	0.028
540	69.636	0.256	0.499	1.873	0.044	0.357	0.33	0.009	0.003	0.044
600	69.62	0.285	0.518	1.879	0.047	0.389	0.349	0.012	0.003	0.056
660	69.605	0.313	0.537	1.904	0.047	0.421	0.374	0.012	-0.149	0.075
720	69.573	0.342	0.556	1.917	0.056	0.453	0.393	0.022	-0.139	0.088
780	69.542	0.361	0.575	1.93	0.063	0.484	0.412	0.025	-0.133	0.107
840	69.495	0.38	0.594	1.943	0.056	0.516	0.425	0.022	-0.136	0.117
900	69.479	0.409	0.607	1.955	0.063	0.542	0.438	0.028	-0.13	0.132
960	69.463	0.428	0.619	1.968	0.06	0.567	0.457	0.025	-0.136	0.142
1020	69.432	0.447	0.638	1.974	0.056	0.593	0.469	0.025	-0.133	0.151
1080	69.4	0.456	0.645	1.981	0.053	0.612	0.482	0.019	-0.139	0.158
1140	69.369	0.466	0.657	1.994	0.066	0.638	0.501	0.028	-0.13	0.167
1200	69.337	0.475	0.67	2.006	0.075	0.657	0.507	0.041	-0.117	0.177
1260	69.306	0.485	0.676	2.013	0.075	0.676	0.52	0.044	-0.114	0.183
1320	69.275	0.504	0.695	2.025	0.094	0.695	0.533	0.066	-0.092	0.196
1380	69.243	0.523	0.708	2.038	0.088	0.714	0.539	0.066	-0.092	0.205
1440	69.212	0.542	0.72	2.057	0.107	0.733	0.558	0.089	-0.073	0.224
1680	69.039	0.589	0.765	2.083	0.085	0.797	0.584	0.073	-0.082	0.256
1920	68.945	0.608	0.777	2.095	0.075	0.848	0.628	0.038	-0.127	0.256
2160	68.662	0.637	0.803	2.115	0.097	0.874	0.647	0.076	-0.085	0.284
2400	68.316	0.656	0.822	2.134	0.091	0.906	0.672	0.073	-0.088	0.306
2640	68.033	0.665	0.828	2.134	0.072	0.925	0.691	0.051	-0.111	0.306
2880	67.64	0.694	0.86	2.172	0.123	0.95	0.711	0.098	-0.063	0.335
3120	67.373	0.732	0.904	2.197	0.085	0.976	0.723	0.063	-0.101	0.354
3360	67.059	0.751	0.916	2.21	0.085	1.008	0.755	0.041	-0.117	0.357
3600	66.933	0.76	0.935	2.236	0.11	1.02	0.768	0.082	-0.076	0.382
3840	66.918	0.789	0.961	2.248	0.094	1.04	0.78	0.076	-0.079	0.401
4080	66.949	0.789	0.961	2.248	0.091	1.052	0.793	0.063	-0.095	0.401
4320	66.918	0.818	0.986	2.267	0.12	1.071	0.812	0.105	-0.053	0.417

CAMP LEJEUNE AQUIFER TEST
PROJECT NUMBER 16032 WBS 0293001
DRW-1 (DEEP WELL) 24 HOUR RECOVERY TEST FOLLOWING 72 HOUR PUMPING TEST
FEBRUARY 23-24, 1996
DRAWDOWN IN FEET

ELAPSED TIME IN MINUTES	DRW-1	6GW1DB	6GW1DA	6GW1D	6GW1S	DP2	DP1	SP2	SRW-1	6GW15D
0	66.902	0.818	0.986	2.267	0.12	1.071	0.806	0.102	-0.057	0.417
0	66.902	0.818	0.986	2.267	0.12	1.071	0.806	0.102	-0.057	0.417
0.5	66.792	0.808	0.98	2.267	0.12	1.065	0.806	0.102	-0.057	0.414
1	59.186	0.808	0.973	2.255	0.12	1.065	0.799	0.102	-0.06	0.411
1.5	44.422	0.799	0.973	2.255	0.12	1.065	0.806	0.102	-0.057	0.414
2	35.172	0.799	0.973	2.236	0.12	1.059	0.806	0.105	-0.057	0.411
2.5	28.17	0.799	0.98	2.216	0.123	1.059	0.806	0.105	-0.057	0.414
3	22.583	0.808	0.98	2.185	0.123	1.065	0.806	0.102	-0.053	0.414
3.5	18.08	0.808	0.98	2.153	0.123	1.065	0.806	0.102	-0.053	0.414
4	14.632	0.818	0.986	2.121	0.126	1.071	0.812	0.105	-0.05	0.414
4.5	11.735	0.818	0.992	2.076	0.129	1.065	0.812	0.105	-0.05	0.42
5	9.451	0.818	0.986	2.025	0.126	1.065	0.806	0.102	-0.053	0.414
5.5	7.703	0.808	0.986	1.974	0.126	1.065	0.806	0.102	-0.057	0.417
6	6.395	0.818	0.992	1.93	0.129	1.065	0.806	0.102	-0.05	0.417
6.5	5.293	0.808	0.986	1.879	0.132	1.065	0.799	0.105	-0.053	0.414
7	4.458	0.818	0.992	1.834	0.132	1.065	0.799	0.098	-0.057	0.414
7.5	3.812	0.818	0.992	1.79	0.129	1.065	0.799	0.098	-0.06	0.411
8	3.324	0.827	1.005	1.751	0.129	1.071	0.806	0.098	-0.057	0.42
8.5	2.914	0.827	1.005	1.713	0.129	1.071	0.806	0.105	-0.057	0.417
9	2.552	0.837	1.011	1.675	0.129	1.071	0.806	0.102	-0.057	0.42
9.5	2.284	0.837	1.011	1.643	0.132	1.065	0.806	0.102	-0.06	0.417
10	2.095	0.846	1.011	1.605	0.129	1.065	0.806	0.102	-0.057	0.417
11	1.796	0.846	1.018	1.548	0.132	1.071	0.806	0.105	-0.053	0.417
12	1.591	0.846	1.024	1.497	0.132	1.071	0.806	0.105	-0.053	0.417
13	1.449	0.856	1.03	1.446	0.129	1.071	0.799	0.105	-0.053	0.414
14	1.323	0.856	1.03	1.401	0.129	1.071	0.799	0.105	-0.053	0.411
15	1.228	0.846	1.024	1.357	0.126	1.065	0.787	0.098	-0.06	0.411
20	0.945	0.856	1.037	1.21	0.123	1.065	0.774	0.102	-0.06	0.398
25	0.787	0.865	1.03	1.108	0.123	1.065	0.768	0.102	-0.063	0.385
30	0.677	0.875	1.024	1.038	0.123	1.065	0.755	0.105	-0.06	0.373
35	0.582	0.884	1.018	0.981	0.123	1.065	0.749	0.105	-0.053	0.36
40	0.519	0.884	1.005	0.936	0.12	1.065	0.749	0.108	-0.053	0.348
45	0.472	0.894	0.992	0.898	0.12	1.065	0.742	0.105	-0.053	0.338
50	0.425	0.884	0.973	0.866	0.113	1.065	0.736	0.102	-0.057	0.325
55	0.378	0.894	0.961	0.834	0.113	1.065	0.73	0.102	-0.06	0.319
60	0.346	0.894	0.942	0.809	0.11	1.059	0.73	0.108	-0.05	0.303
90	0.22	0.875	0.86	0.707	0.091	1.059	0.711	0.086	-0.066	0.246
120	0.157	0.875	0.796	0.643	0.088	1.059	0.704	0.086	-0.069	0.205
150	0.11	0.837	0.733	0.592	0.079	1.046	0.698	0.089	-0.139	0.167
180	0.063	0.808	0.683	0.547	0.072	1.014	0.685	0.092	19.553	0.132
210	0.031	0.779	0.651	0.522	0.066	0.982	0.704	0.095	19.575	0.113
240	0.015	0.741	0.613	0.497	0.056	0.931	0.679	0.066	19.556	0.085
270	0	0.732	0.588	0.484	0.047	0.893	0.679	0.044	2.536	0.06
300	-0.031	0.684	0.543	0.446	0.034	0.848	0.666	0.038	2.374	0.037
360	-0.078	0.608	0.486	0.395	0.018	0.784	0.653	0.012	2.336	0
420	-0.11	0.561	0.442	0.369	0.028	0.727	0.647	0.009	2.336	0
480	-0.126	0.513	0.411	0.337	0.025	0.682	0.647	0.015	2.339	0
540	-0.141	0.475	0.379	0.312	0.037	0.644	0.641	0.028	2.349	0
600	-0.173	0.437	0.347	0.293	0.028	0.606	0.628	0.025	2.345	0
660	-0.189	0.399	0.322	0.274	0.028	0.574	0.622	0.028	2.349	0
720	-0.204	0.361	0.297	0.254	0.041	0.542	0.615	0.044	2.364	0
780	-0.22	0.332	0.278	0.235	0.041	0.516	0.609	0.051	2.371	0
840	-0.236	0.304	0.252	0.223	0.012	0.497	0.603	0.028	2.349	0
900	-0.252	0.285	0.24	0.203	0.015	0.478	0.603	0.025	2.345	0
960	-0.267	0.256	0.221	0.191	0.006	0.459	0.596	0.012	2.336	0
1020	-0.267	0.247	0.208	0.178	0.018	0.446	0.596	0.022	2.345	0
1080	-0.283	0.228	0.189	0.165	-0.006	0.427	0.59	0.003	2.326	0
1140	-0.299	0.199	0.17	0.146	-0.018	0.408	0.584	-0.015	2.307	0
1200	-0.315	0.18	0.158	0.133	-0.009	0.395	0.584	-0.012	2.307	0
1260	-0.33	0.161	0.139	0.121	-0.015	0.376	0.584	-0.025	2.298	0
1320	-0.33	0.142	0.126	0.114	-0.006	0.363	0.577	-0.022	2.301	0
1380	-0.33	0.142	0.126	0.114	0.006	0.35	0.571	-0.006	2.317	0
1440	-0.33	0.133	0.126	0.114	0.015	0.338	0.577	0.006	2.323	0

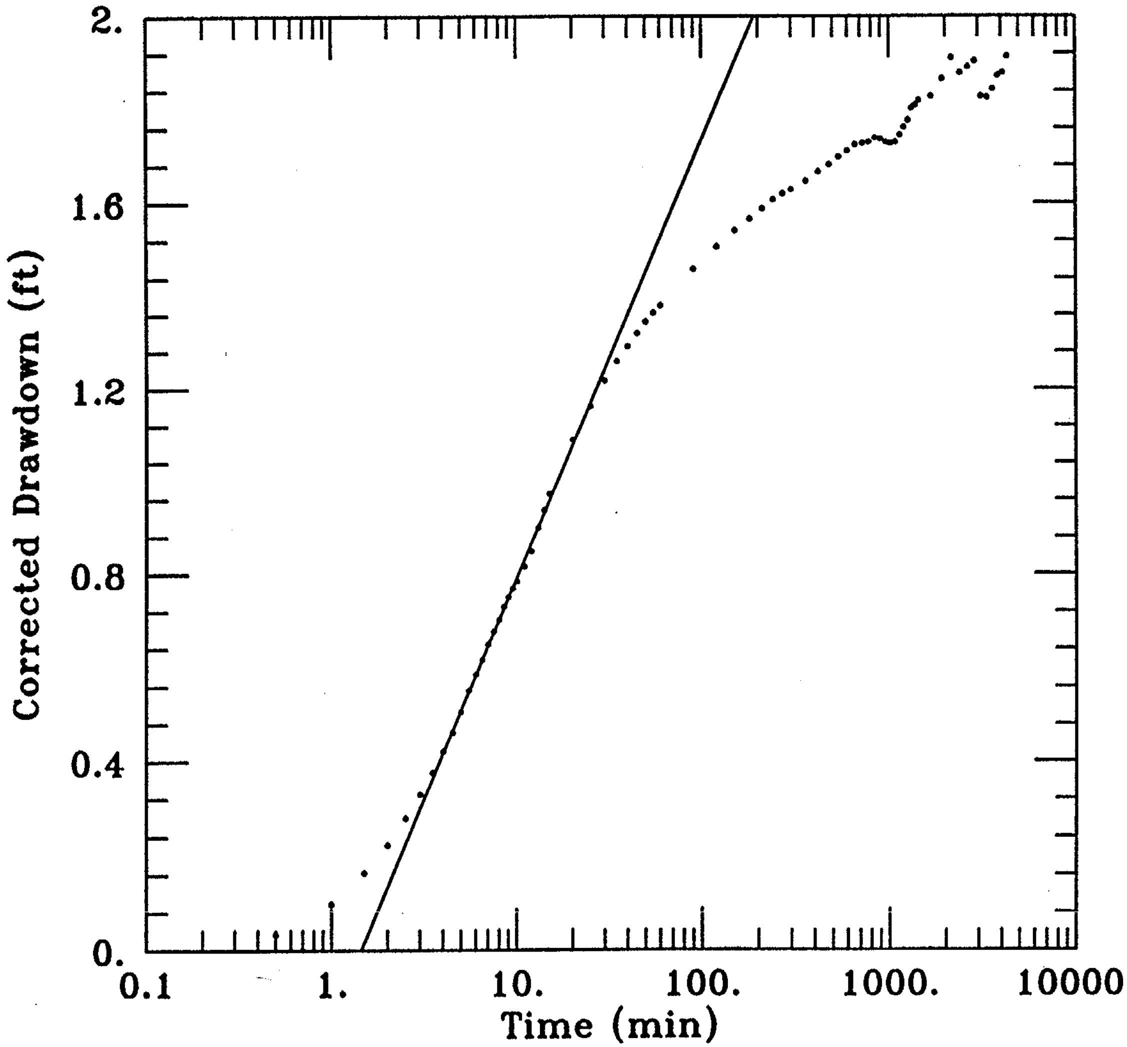
CAMP LEJEUNE AQUIFER TEST
PROJECT NUMBER 16032 WBS 0293001
DRW-1 (DEEP WELL) STEP DRAWDOWN TEST
FEBRUARY 19, 1996
FLOW RATE: 30 GPM
DRAWDOWN IN FEET

ELAPSED TIME IN MINUTES	DRW-1	6GW1DB	6GW1DA	6GW1D	6GW1S	DP2	DP1	SP2	SRW-1	6GW15D
0	0.063	-0.019	-0.006	0.178	-0.009	0.351	0.177	0.038	0.625	0
0.5	10.301	-0.019	-0.006	0.172	-0.006	0.344	0.177	0.035	0.625	0
1	29.68	-0.009	-0.006	0.172	-0.012	0.351	0.165	0.028	0.622	0
1.5	43.698	-0.019	0	0.178	-0.012	0.344	0.165	0.028	0.619	0
2	54.139	-0.009	0	0.203	-0.012	0.351	0.177	0.028	0.622	0
2.5	60.772	-0.009	0.006	0.248	-0.009	0.363	0.196	0.044	0.632	0
3	63.444	-0.009	0.006	0.305	-0.012	0.363	0.203	0.054	0.641	0
3.5	65.157	-0.009	0	0.363	-0.012	0.363	0.203	0.051	0.638	0
4	66.257	-0.009	0	0.426	-0.015	0.357	0.203	0.047	0.635	0
4.5	66.838	-0.028	-0.012	0.477	-0.018	0.351	0.184	0.044	0.632	0
5	66.948	-0.019	-0.012	0.541	-0.022	0.357	0.177	0.044	0.632	0
5.5	67.042	-0.028	-0.012	0.598	-0.022	0.351	0.177	0.047	0.638	0
6	67.137	-0.019	-0.006	0.656	-0.022	0.363	0.19	0.051	0.641	0
6.5	67.247	-0.028	-0.012	0.694	-0.018	0.351	0.184	0.047	0.638	0
7	67.325	-0.038	-0.018	0.739	-0.022	0.351	0.19	0.044	0.635	0
7.5	67.404	-0.019	-0.006	0.783	-0.022	0.357	0.196	0.038	0.632	0
8	67.498	-0.038	-0.018	0.809	-0.025	0.344	0.19	0.038	0.628	0
8.5	67.577	-0.038	-0.018	0.834	-0.025	0.351	0.19	0.041	0.632	0
9	67.64	-0.038	-0.025	0.866	-0.025	0.351	0.196	0.044	0.635	0
9.5	67.718	-0.038	-0.025	0.892	-0.025	0.351	0.196	0.041	0.632	0
10	67.734	-0.038	-0.025	0.923	-0.025	0.357	0.203	0.041	0.628	0
11	67.86	-0.028	-0.025	0.962	-0.025	0.363	0.203	0.035	0.628	0
12	68.001	-0.047	-0.031	1	-0.025	0.357	0.19	0.038	0.632	0
13	68.08	-0.038	-0.025	1.032	-0.025	0.363	0.196	0.041	0.635	0
14	68.205	-0.057	-0.037	1.057	-0.025	0.351	0.184	0.025	0.619	0
15	68.268	-0.057	-0.037	1.089	-0.025	0.357	0.19	0.022	0.619	0
20	68.677	-0.066	-0.037	1.191	-0.018	0.363	0.203	0.035	0.635	0
25	68.912	-0.057	-0.025	1.267	-0.022	0.37	0.222	0.025	0.622	0
30	69.116	-0.057	-0.012	1.331	-0.015	0.389	0.241	0.044	0.632	0
35	69.274	-0.085	-0.012	1.369	-0.018	0.382	0.241	0.028	0.616	0
40	69.321	-0.085	0	1.395	-0.015	0.389	0.241	0.038	0.628	0
45	69.352	-0.076	0.018	1.433	-0.015	0.402	0.247	0.035	0.625	0
50	69.368	-0.085	0.025	1.458	-0.015	0.408	0.266	0.031	0.619	0
55	69.368	-0.085	0.044	1.484	-0.012	0.421	0.279	0.031	0.622	0
60	69.368	-0.066	0.069	1.516	-0.012	0.433	0.292	0.035	0.625	0
90	69.368	-0.066	0.132	1.586	-0.012	0.459	0.292	0.022	0.616	0

Attachment E – Analytical Solution and Graphs used in the Cooper-Jacob Method for the Shallow Pumping Test Data

Client: **LANTDIV**	Company: **Baker Environmental, Inc.**
Location: **Site 82 -- MCB Camp Lejeune**	Project: **62470-222**

SP-2 DRAWDOWN DATA



DATA SET:
SP-2D.DAT
03/06/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Cooper-Jacob

PROJECT DATA:
test date: February 26-29, 1996
test well: SRW-1
obs. well: SP-2

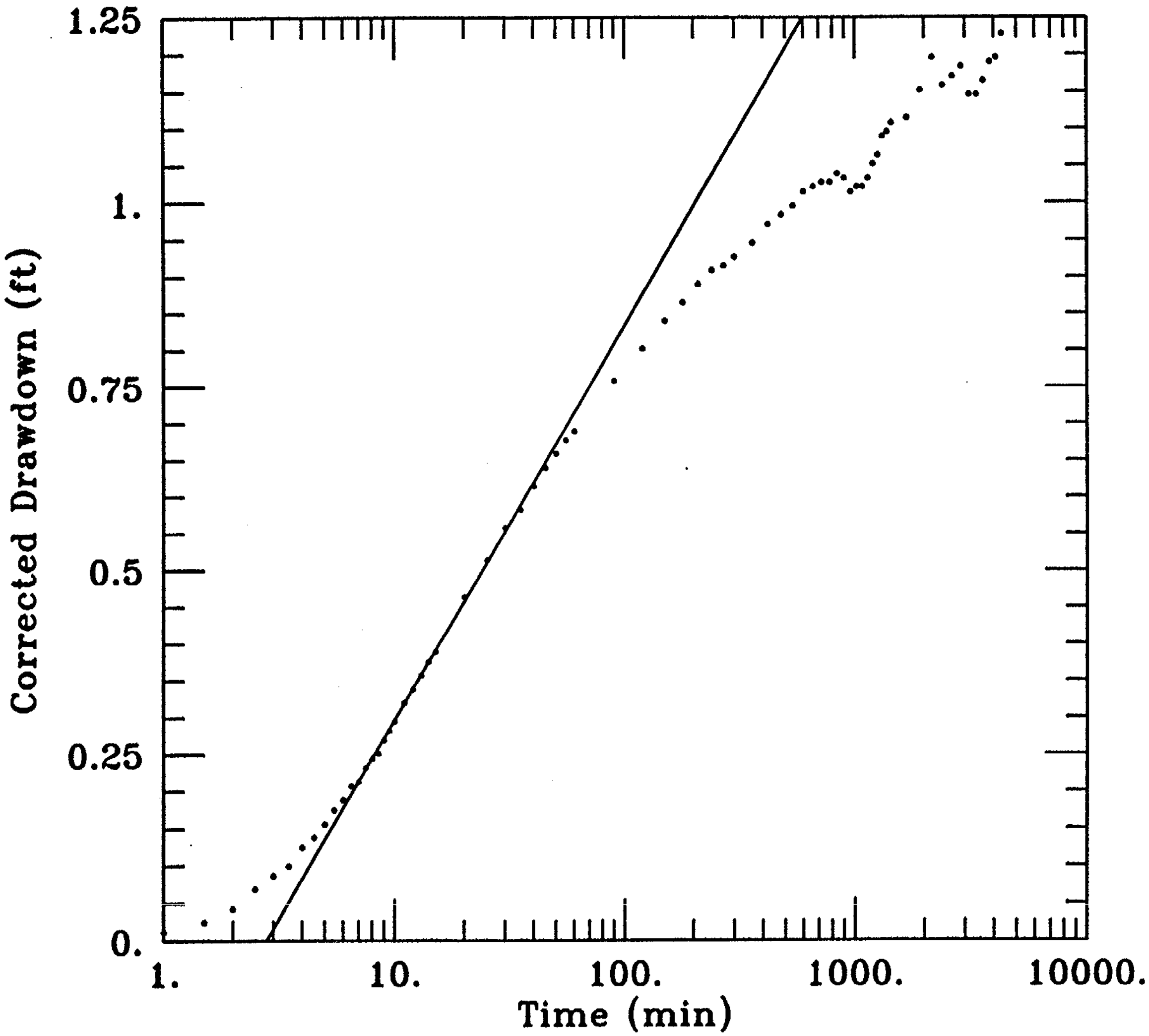
TEST DATA:
Q = 5. gal/min
r = 13.15 ft
r_c = 0.125 ft
r_w = 0.25 ft
b = 300. ft

PARAMETER ESTIMATES:
T = 187.1 ft^2/day
S = 0.002432

AQTESOLV

Client: **LANTDIV**	Company: **Baker Environmental, Inc.**
Location: **Site 82 -- MCB Camp Lejeune**	Project: **62470-222**

SP-1 DRAWDOWN DATA



DATA SET:
SP-1D.DAT
03/06/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Cooper-Jacob

PROJECT DATA:
test date: February 26-29, 1996
test well: SRW-1
obs. well: SP-1

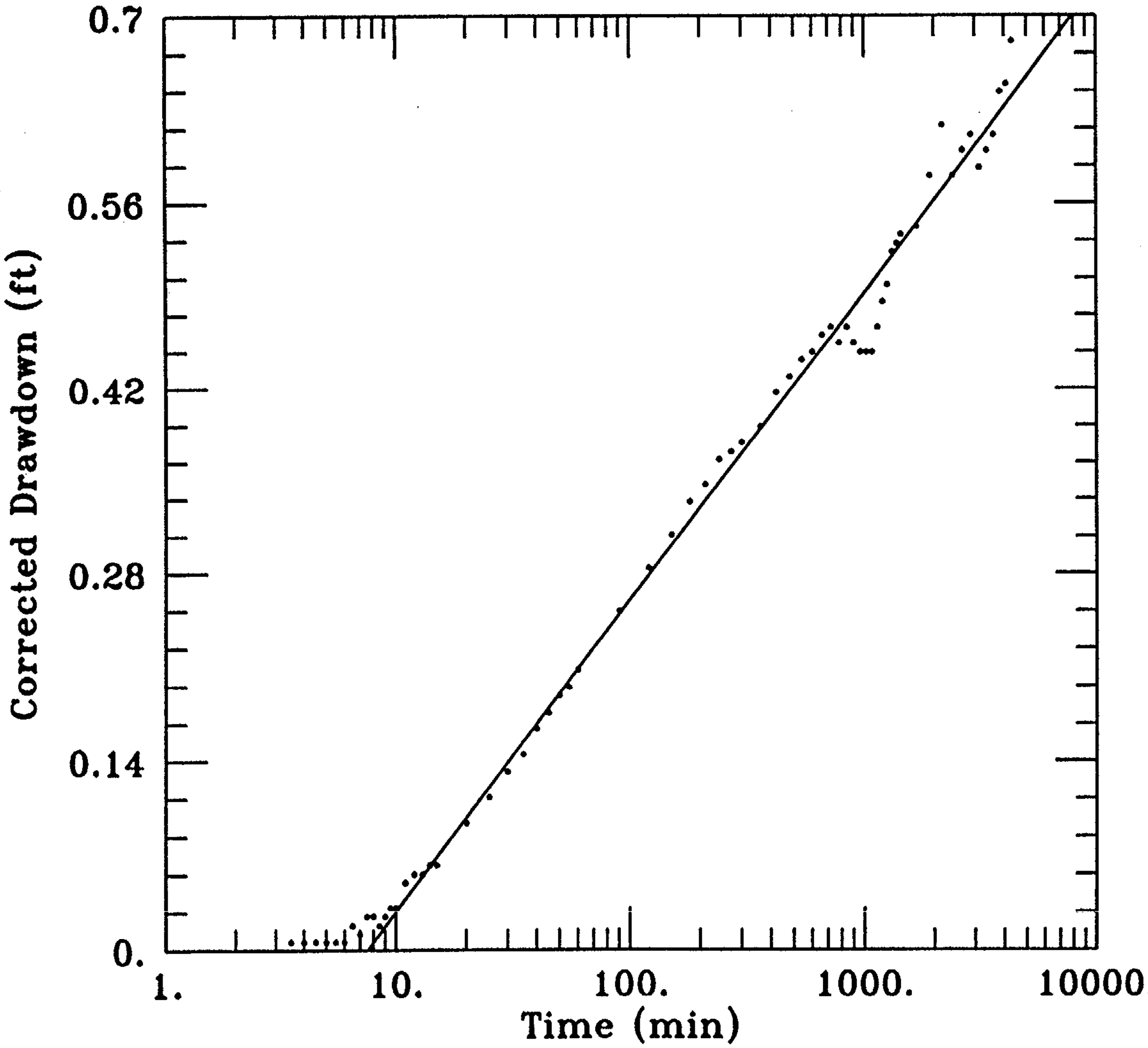
TEST DATA:
Q = 5. gal/min
r = 16.64 ft
r_c = 0.125 ft
r_w = 0.25 ft
b = 300. ft

PARAMETER ESTIMATES:
T = 328.7 ft^2/day
S = 0.005182

AQTESOLV

Client: **LANTDIV**	Company: **Baker Environmental, Inc.**
Location: **Site 82 -- MCB Camp Lejeune**	Project: **62470-222**

6GW-34 DRAWDOWN DATA



DATA SET:
34-D.DAT
03/08/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Cooper-Jacob

PROJECT DATA:
test date: February 26-29, 1996
test well: SRW-1
obs. well: 6GW-34

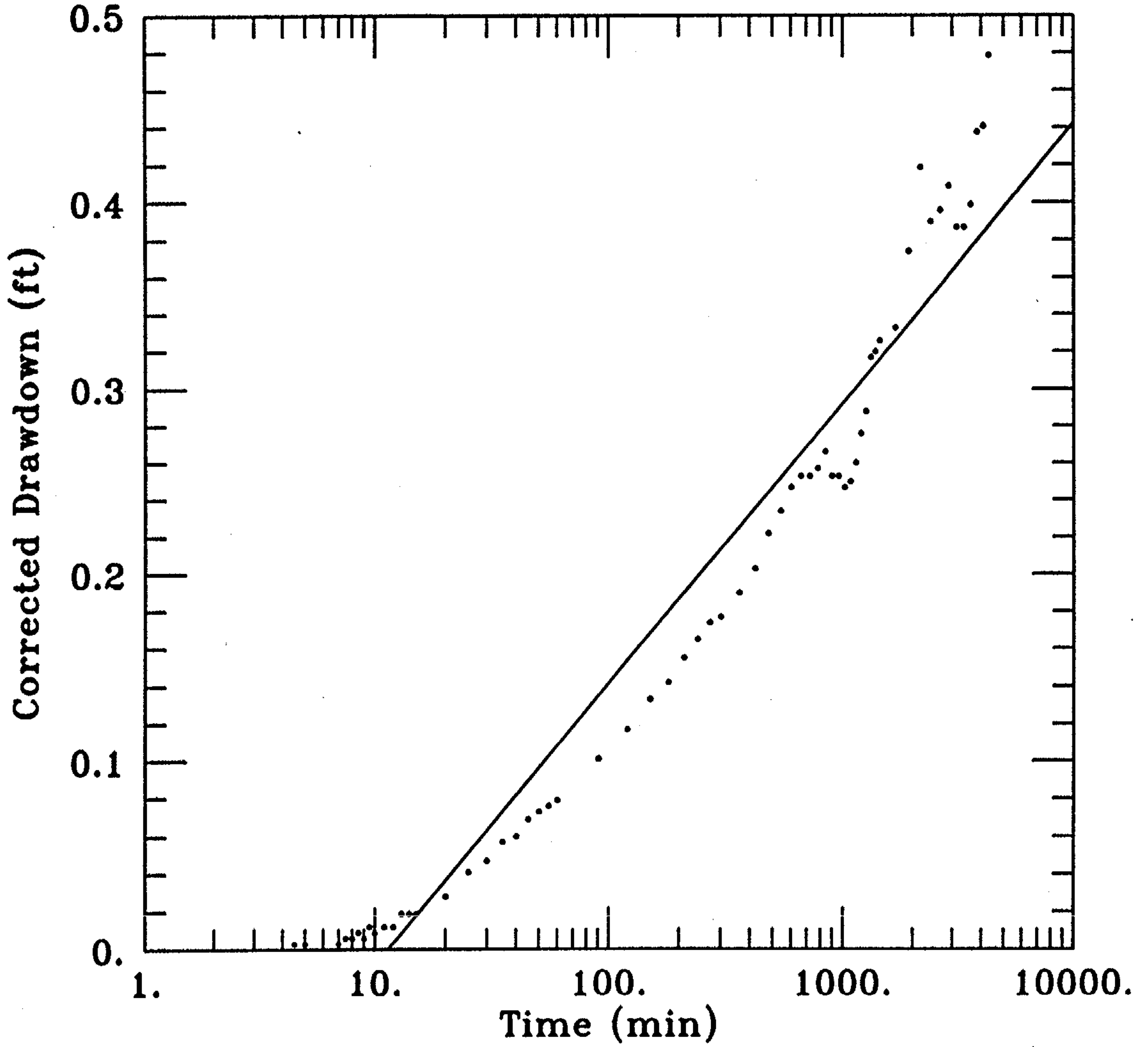
TEST DATA:
Q = 5. gal/min
r = 96.27 ft
r_c = 0.125 ft
r_w = 0.25 ft
b = 300. ft

PARAMETER ESTIMATES:
T = 760.7 ft^2/day
S = 0.0009688

AQTESOLV

Client: **LANTDIV**	Company: **Baker Environmental, Inc.**
Location: **Site 82 -- MCB Camp Lejeune**	Project: **62470-222**

SP-3 DRAWDOWN DATA



DATA SET:
SP-3D.DAT
03/06/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Cooper-Jacob

PROJECT DATA:
test date: February 26-29, 1996
test well: SRW-1
obs. well: SP-3

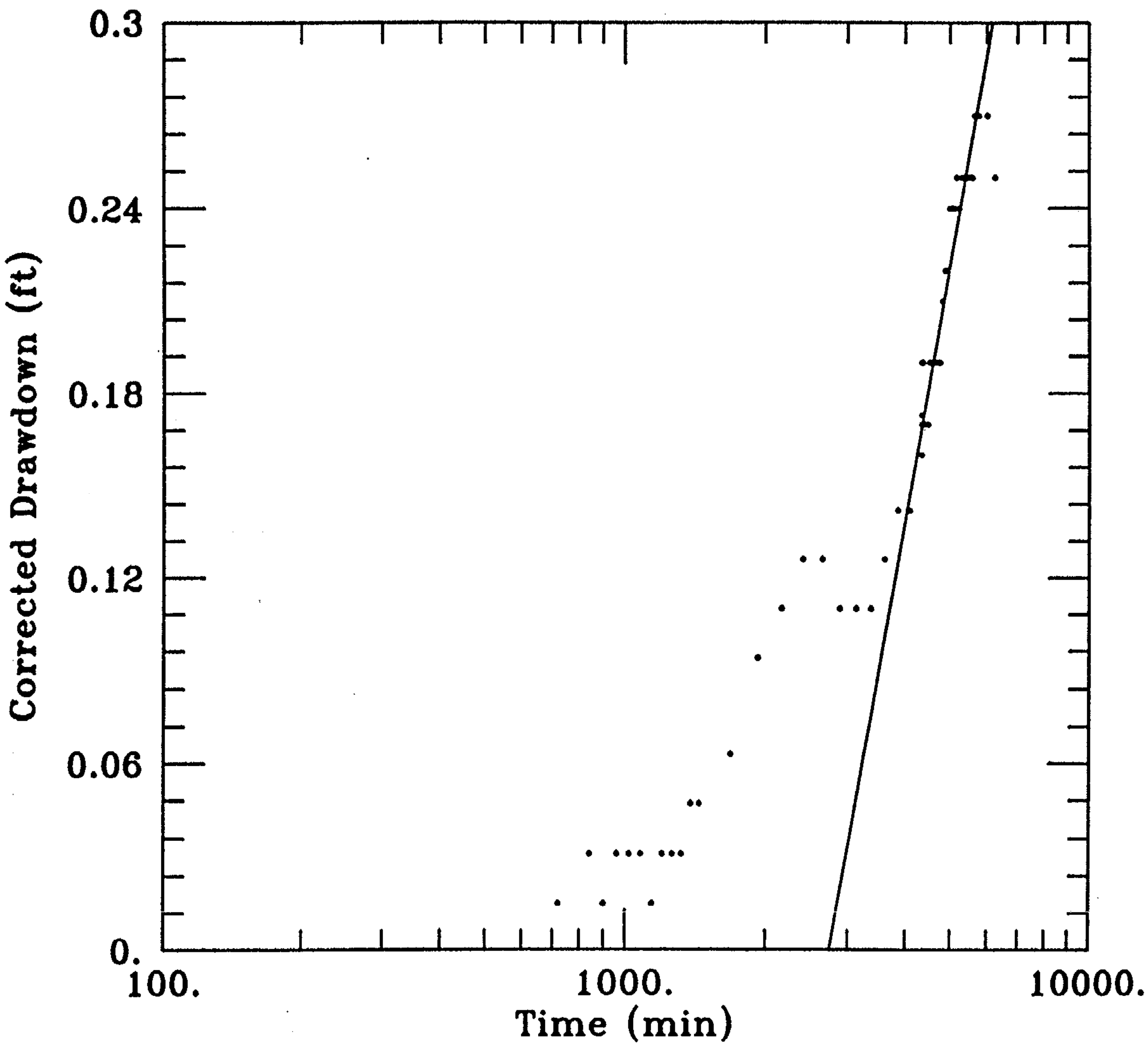
TEST DATA:
Q = 5. gal/min
r = 93.74 ft
r_c = 0.125 ft
r_w = 0.25 ft
b = 300. ft

PARAMETER ESTIMATES:
T = 1174.3 ft^2/day
S = 0.002373

AQTESOLV

Client: LANTDIV	Company: Baker Environmental, Inc.
Location: Site 82 -- MCB Camp Lejeune	Project: 62470-222

DRW-1 DRAWDOWN DATA



DATA SET:
DRW-1DR.DAT
03/10/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Cooper-Jacob

PROJECT DATA:
test date: February 26-29, 1996
test well: SRW-1
obs. well: DRW-1

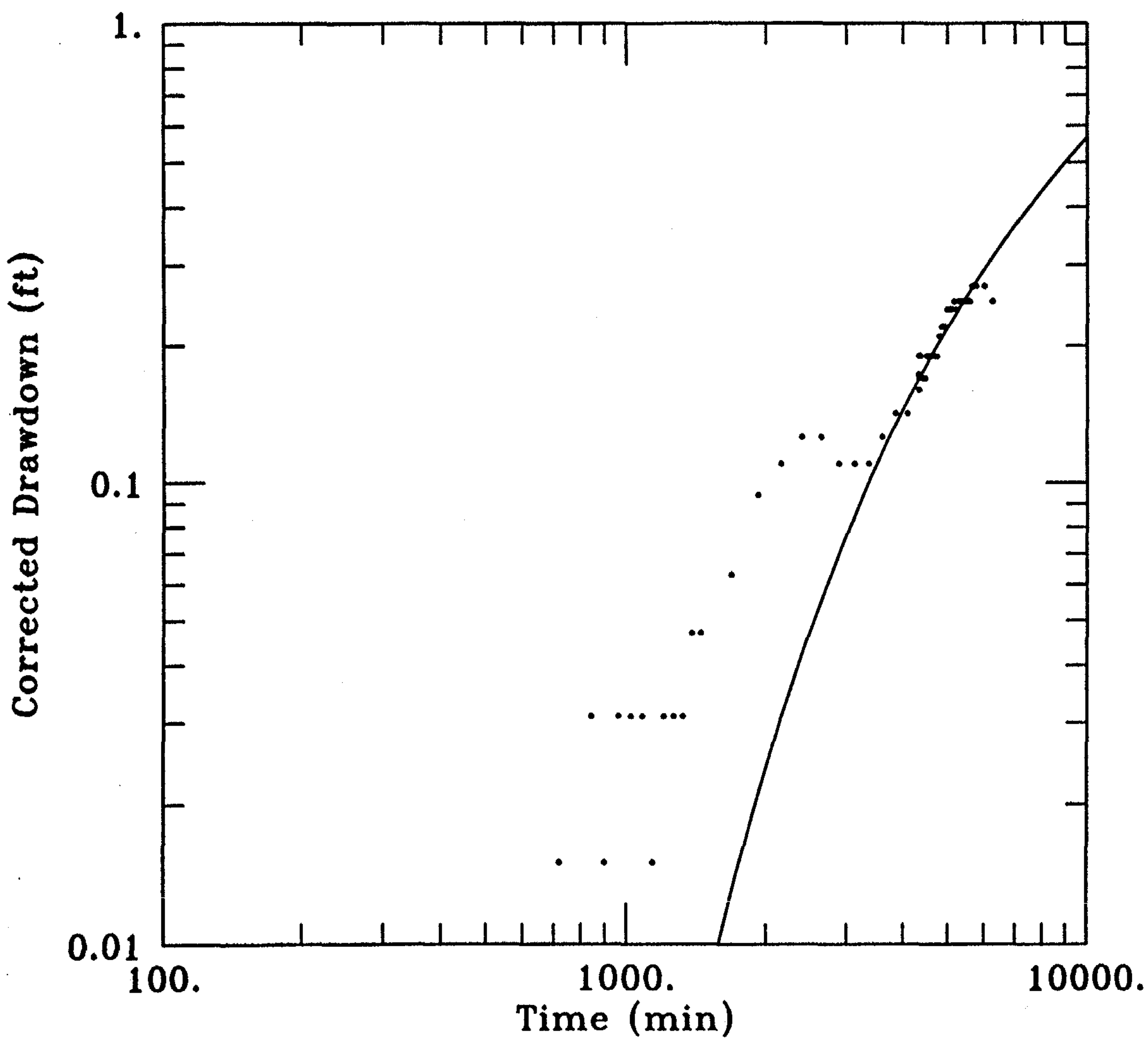
TEST DATA:
Q = 5. gal/min
r = 304.1 ft
r_c = 0.125 ft
r_w = 0.25 ft
b = 300. ft

PARAMETER ESTIMATES:
T = 207.3 ft^2/day
S = 0.009593

AQTESOLV

Client: LANTDIV	Company: Baker Environmental, Inc.
Location: Site 82 -- MCB Camp Lejeune	Project: 62470-222

DRW-1 DRAWDOWN DATA



DATA SET:
DRW-1DR.DAT
03/10/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Theis

PROJECT DATA:
test date: February 26-29, 1996
test well: SRW-1
obs. well: DRW-1

TEST DATA:
Q = 5. gal/min
r = 304.1 ft
r_c = 0.125 ft
r_w = 0.25 ft
b = 300. ft

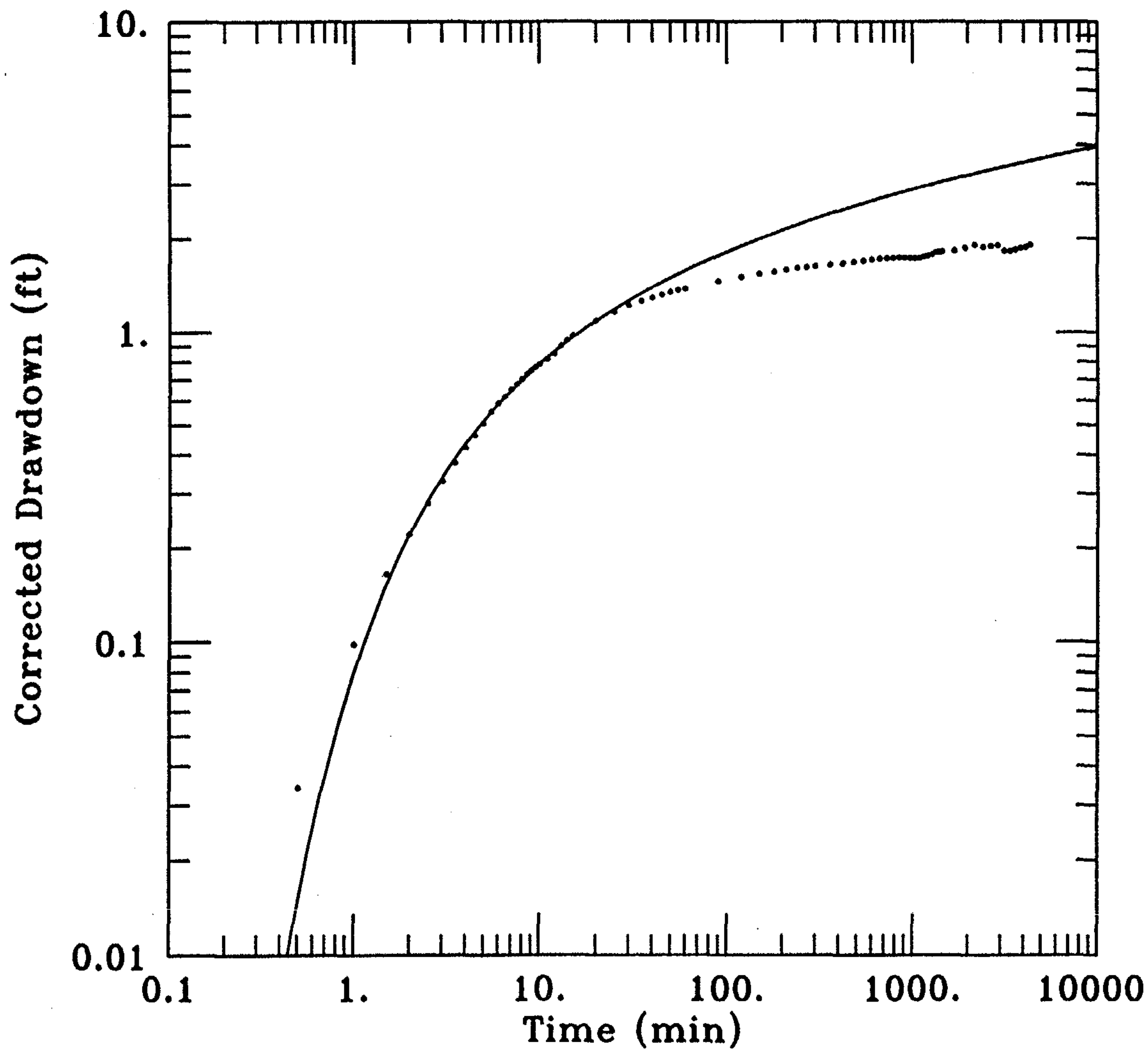
PARAMETER ESTIMATES:
T = 74.11 ft^2/day
S = 0.01136

AQTESOLV

Attachment F – Analytical Solution and Graphs used in the Theis Method for the Shallow Pumping Test Data

Client: LANTDIV	Company: Baker Environmental, Inc.
Location: Site 82 -- MCB Camp Lejeune	Project: 62470-222

SP-2 DRAWDOWN DATA



DATA SET:
SP-2D.DAT
03/06/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Theis

PROJECT DATA:
test date: February 26-29, 1996
test well: SRW-1
obs. well: SP-2

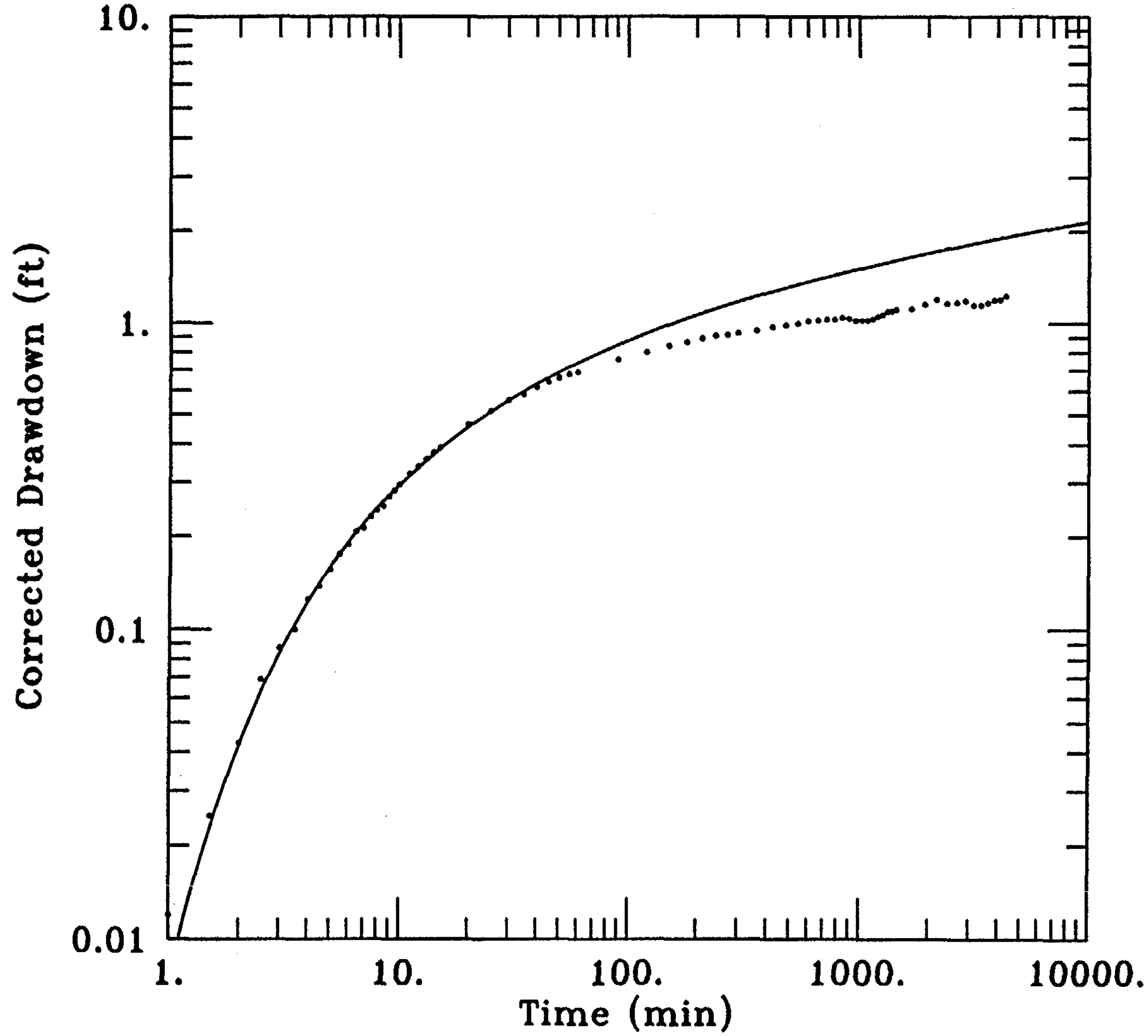
TEST DATA:
Q = 5. gal/min
r = 13.15 ft
r_c= 0.125 ft
r_w= 0.25 ft
b = 300. ft

PARAMETER ESTIMATES:
T = 165. ft^2/day
S = 0.003051

AQTESOLV

Client: **LANTDIV**	Company: **Baker Environmental, Inc.**
Location: **Site 82 -- MCB Camp Lejeune**	Project: **62470-222**

SP-1 DRAWDOWN DATA



DATA SET:
SP-1D.DAT
03/06/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Theis

PROJECT DATA:
test date: February 26-29, 1996
test well: SRW-1
obs. well: SP-1

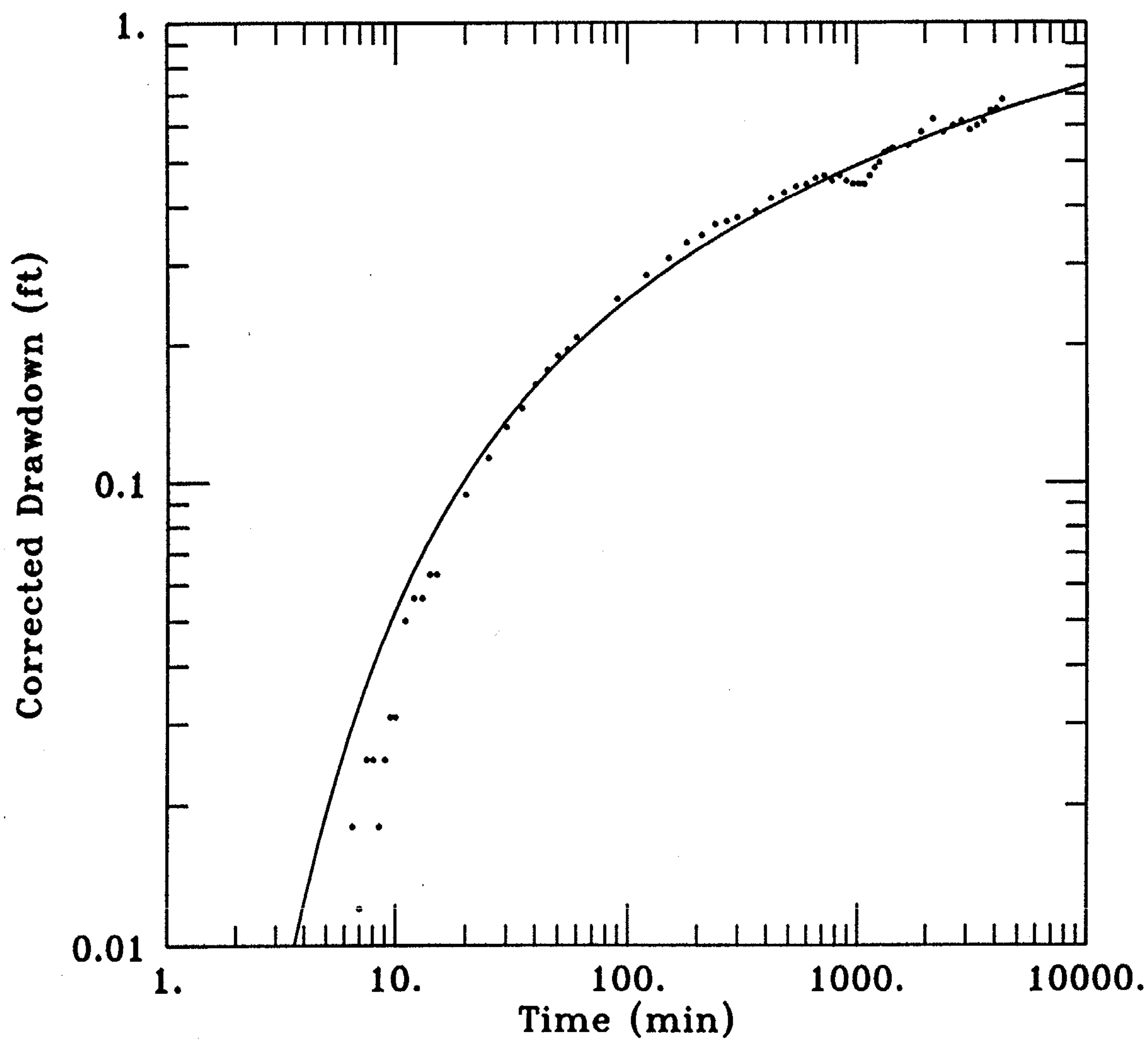
TEST DATA:
Q = 5. gal/min
r = 16.64 ft
r_c = 0.125 ft
r_w = 0.25 ft
b = 300. ft

PARAMETER ESTIMATES:
T = 276.6 ft^2/day
S = 0.006783

AQTESOLV

Client: **LANTDIV**	Company: **Baker Environmental, Inc.**
Location: **Site 82 -- MCB Camp Lejeune**	Project: **62470-222**

6GW-34 DRAWDOWN DATA



DATA SET:
34-D.DAT
03/08/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Theis

PROJECT DATA:
test date: February 26-29, 1996
test well: SRW-1
obs. well: 6GW-34

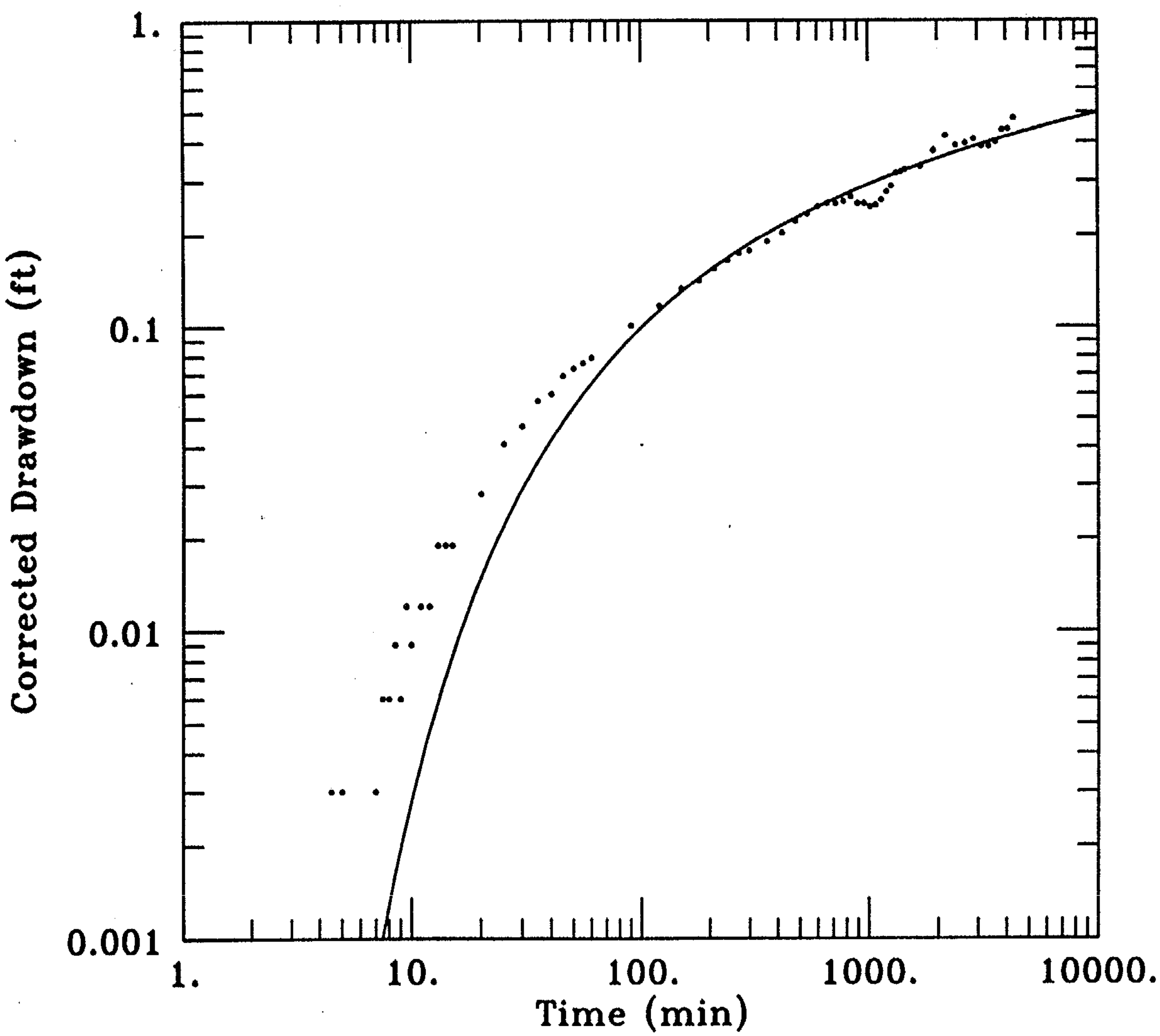
TEST DATA:
Q = 5. gal/min
r = 96.27 ft
r_c = 0.125 ft
r_w = 0.25 ft
b = 300. ft

PARAMETER ESTIMATES:
T = 720.7 ft^2/day
S = 0.001203

AQTESOLV

Client: LANTDIV	Company: Baker Environmental, Inc.
Location: Site 82 -- MCB Camp Lejeune	Project: 62470-222

SP-3 DRAWDOWN DATA



DATA SET:
SP-3D.DAT
03/06/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Theis

PROJECT DATA:
test date: February 26-29, 1996
test well: SRW-1
obs. well: SP-3

TEST DATA:
Q = 5. gal/min
r = 93.74 ft
r_c = 0.125 ft
r_w = 0.25 ft
b = 300. ft

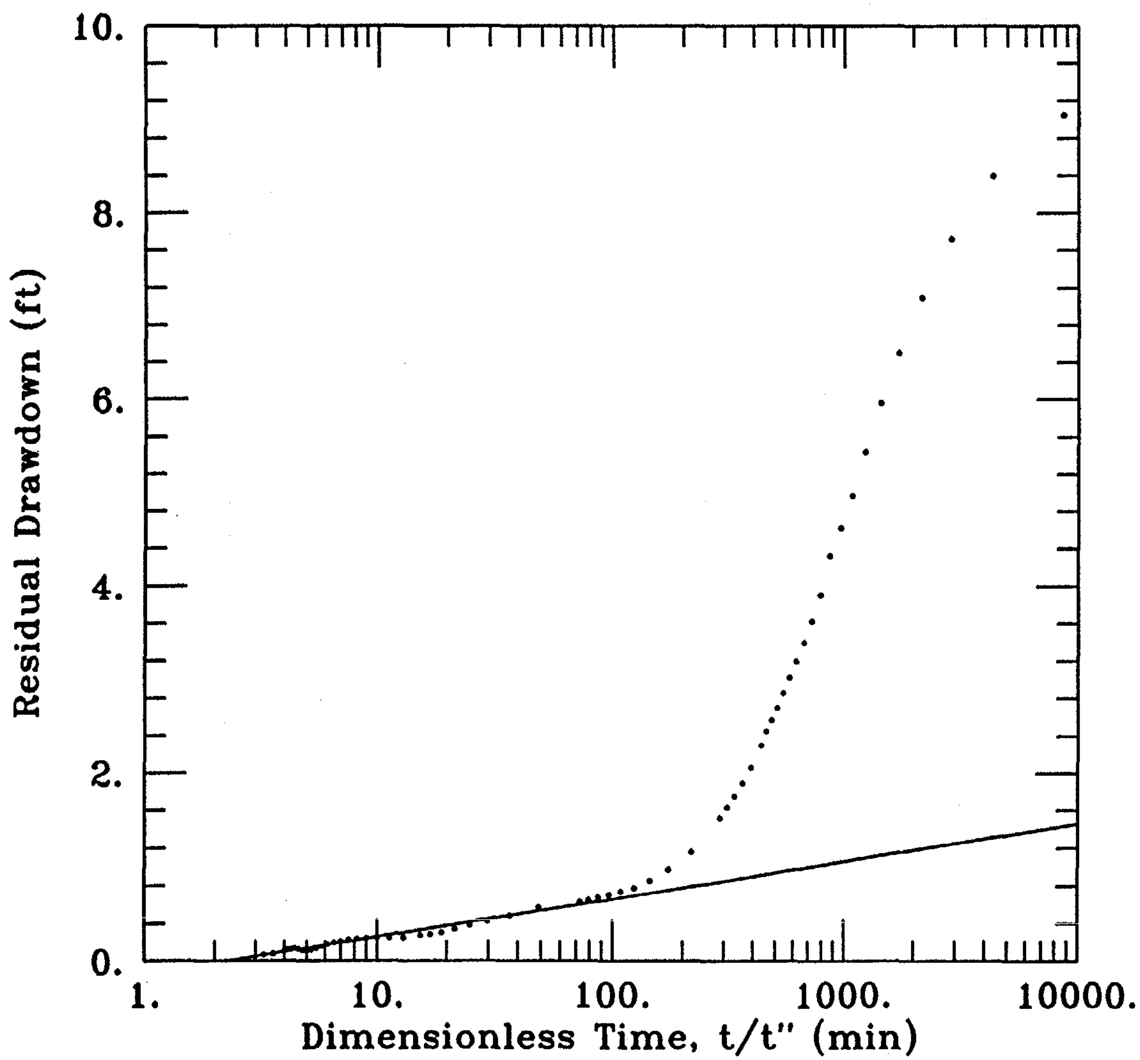
PARAMETER ESTIMATES:
T = 841.6 ft^2/day
S = 0.006247

AQTESOLV

Attachment G – Analytical Solution and Graphs used in the Theis Recovery Method for the Shallow Pumping Test Data

Client: LANTDIV	Company: Baker Environmental, Inc.
Location: Site 82 -- MCB Camp Lejeune	Project: 62470-222

SRW-1 RECOVERY DATA



DATA SET:
SRW-1R.DAT
03/08/96

AQUIFER MODEL:
Confined

SOLUTION METHOD:
Theis Recovery

PROJECT DATA:
test date: February 26-29, 1996
test well: SRW-1
obs. well: SRW-1

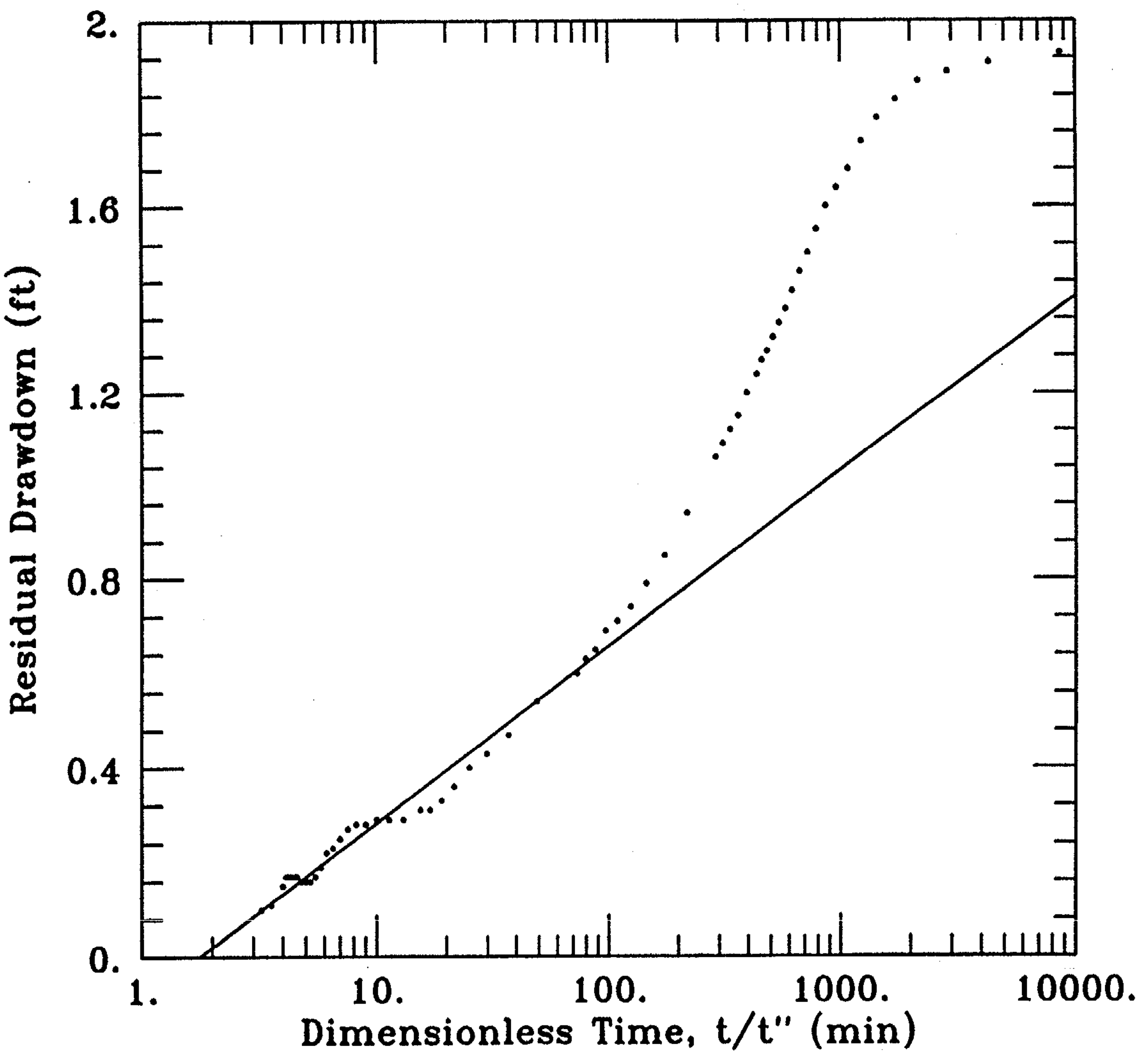
TEST DATA:
Q = 5. gal/min
r = 0. ft
r_c = 0.125 ft
r_w = 0.25 ft
b = 300. ft

PARAMETER ESTIMATES:
T = 441.8 ft^2/day
S' = 2.243

AQTESOLV

Client: LANTDIV	Company: Baker Environmental, Inc.
Location: Site 82 -- MCB Camp Lejeune	Project: 62470-222

SP-2 RECOVERY DATA



DATA SET:
SP-2R.DAT
03/08/96

AQUIFER MODEL:
Confined

SOLUTION METHOD:
Theis Recovery

PROJECT DATA:
test date: February 26-29, 1996
test well: SRW-1
obs. well: SP-2

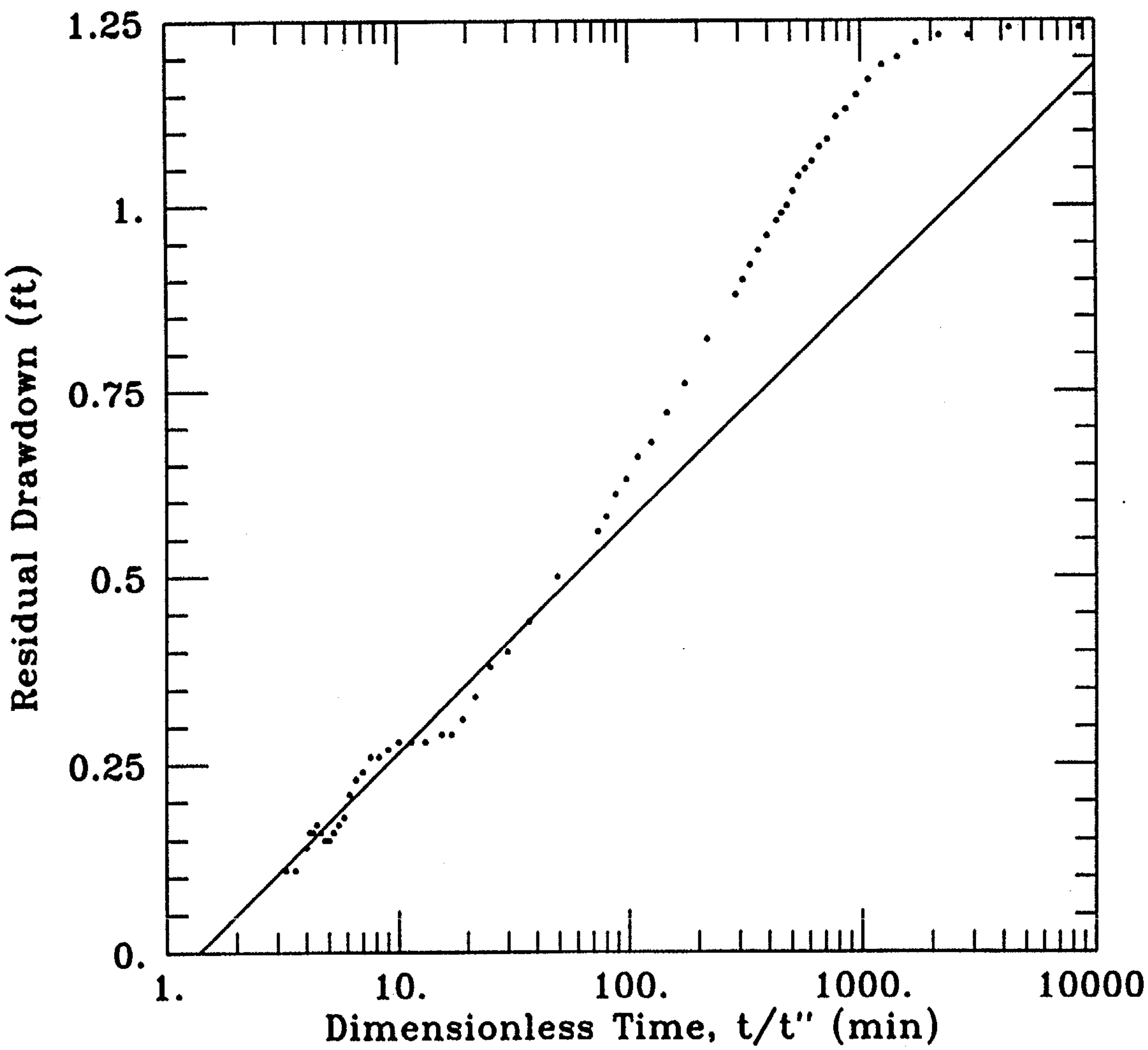
TEST DATA:
Q = 5. gal/min
r = 15.08 ft
r_c = 0.125 ft
r_w = 0.25 ft
b = 300. ft

PARAMETER ESTIMATES:
T = 470.1 ft^2/day
S' = 1.768

AQTESOLV

Client: **LANTDIV**	Company: **Baker Environmental, Inc.**
Location: **Site 82 -- MCB Camp Lejeune**	Project: **62470-222**

SP-1 RECOVERY DATA



DATA SET:
SP-1R.DAT
03/08/96

AQUIFER MODEL:
Confined

SOLUTION METHOD:
Theis Recovery

PROJECT DATA:
test date: February 26-29, 1996
test well: SRW-1
obs. well: SP-1

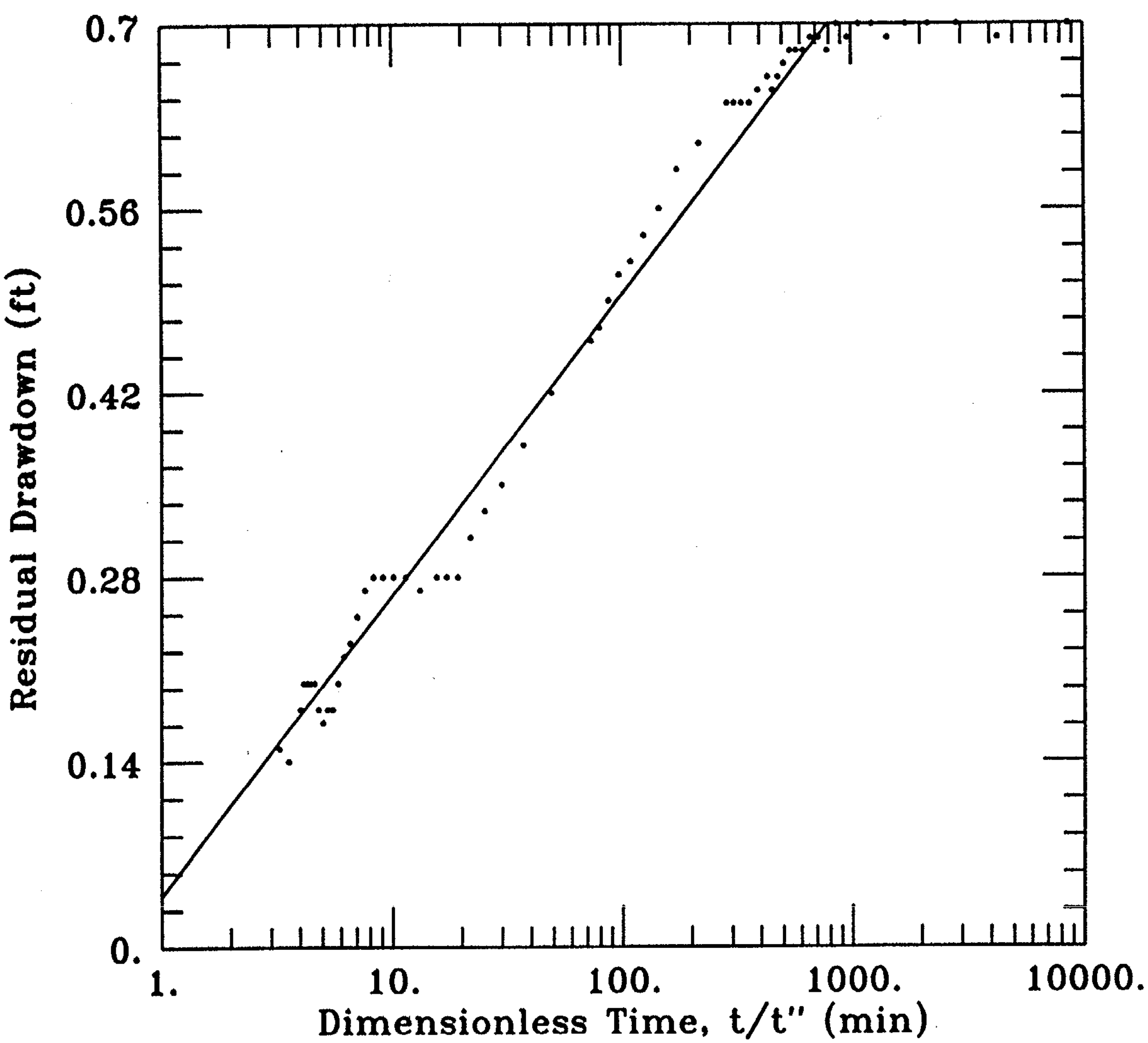
TEST DATA:
Q = 5. gal/min
r = 13.72 ft
r_c= 0.125 ft
r_w= 0.25 ft
b = 300. ft

PARAMETER ESTIMATES:
T = 572. ft^2/day
S' = 1.375

AQTESOLV

Client: LANTDIV	Company: Baker Environmental, Inc.
Location: Site 82 -- MCB Camp Lejeune	Project: 62470-222

6GW-34 RECOVERY DATA



DATA SET:
34-R.DAT
03/08/96

AQUIFER MODEL:
Confined

SOLUTION METHOD:
Theis Recovery

PROJECT DATA:
test date: February 26-29, 1996
test well: SRW-1
obs. well: 6GW-34

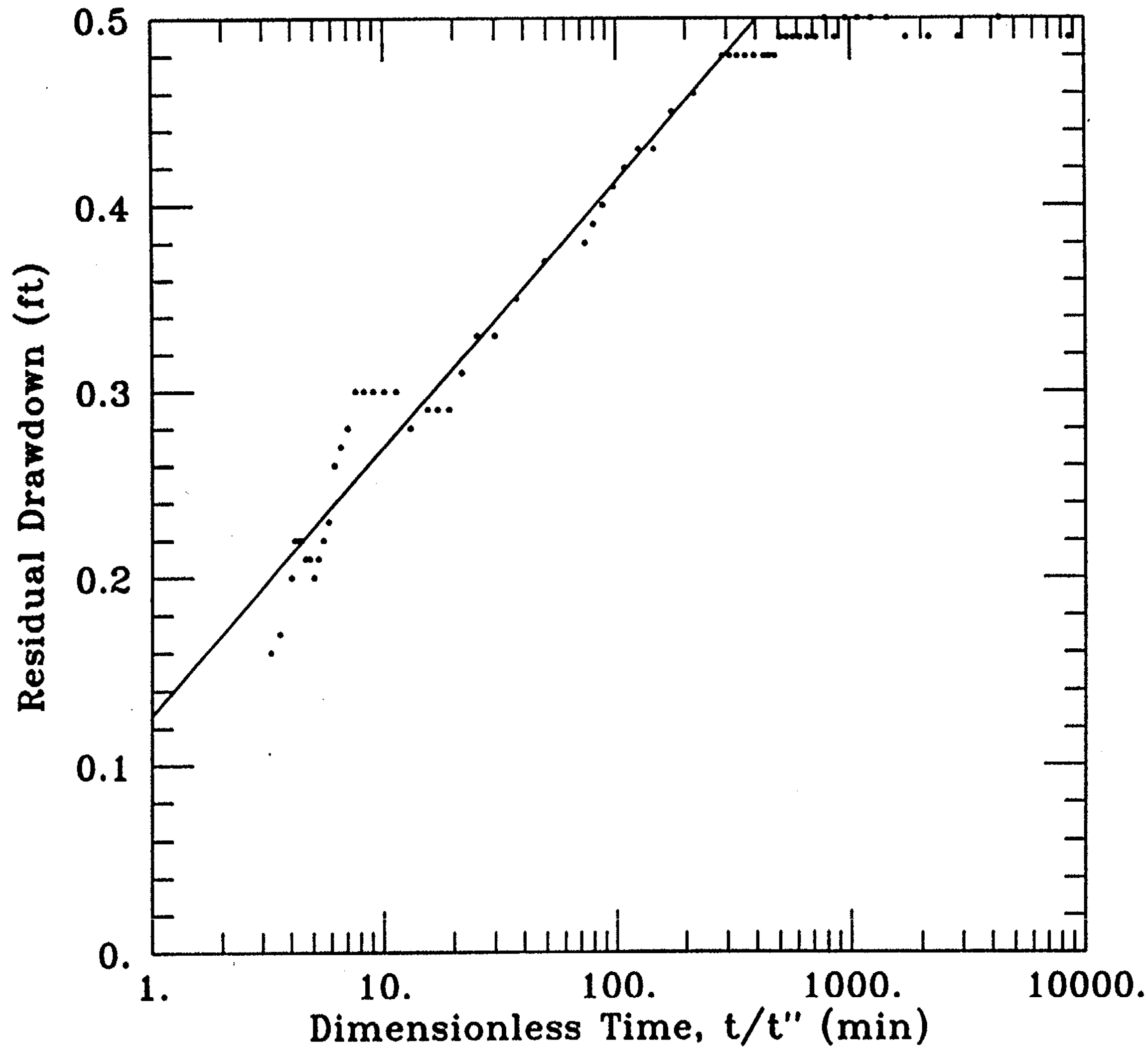
TEST DATA:
Q = 5. gal/min
r = 96.27 ft
r_c = 0.125 ft
r_w = 0.25 ft
b = 300. ft

PARAMETER ESTIMATES:
T = 772.5 ft^2/day
S' = 0.6763

AQTESOLV

Client: **LANTDIV**	Company: **Baker Environmental, Inc.**
Location: **Site 82 -- MCB Camp Lejeune**	Project: **62470-222**

SP-3 RECOVERY DATA



DATA SET:
SP-3R.DAT
03/08/96

AQUIFER MODEL:
Confined

SOLUTION METHOD:
Theis Recovery

PROJECT DATA:
test date: February 26-29, 1996
test well: SRW-1
obs. well: SP-3

TEST DATA:
Q = 5. gal/min
r = 96.29 ft
r_c = 0.125 ft
r_w = 0.25 ft
b = 300. ft

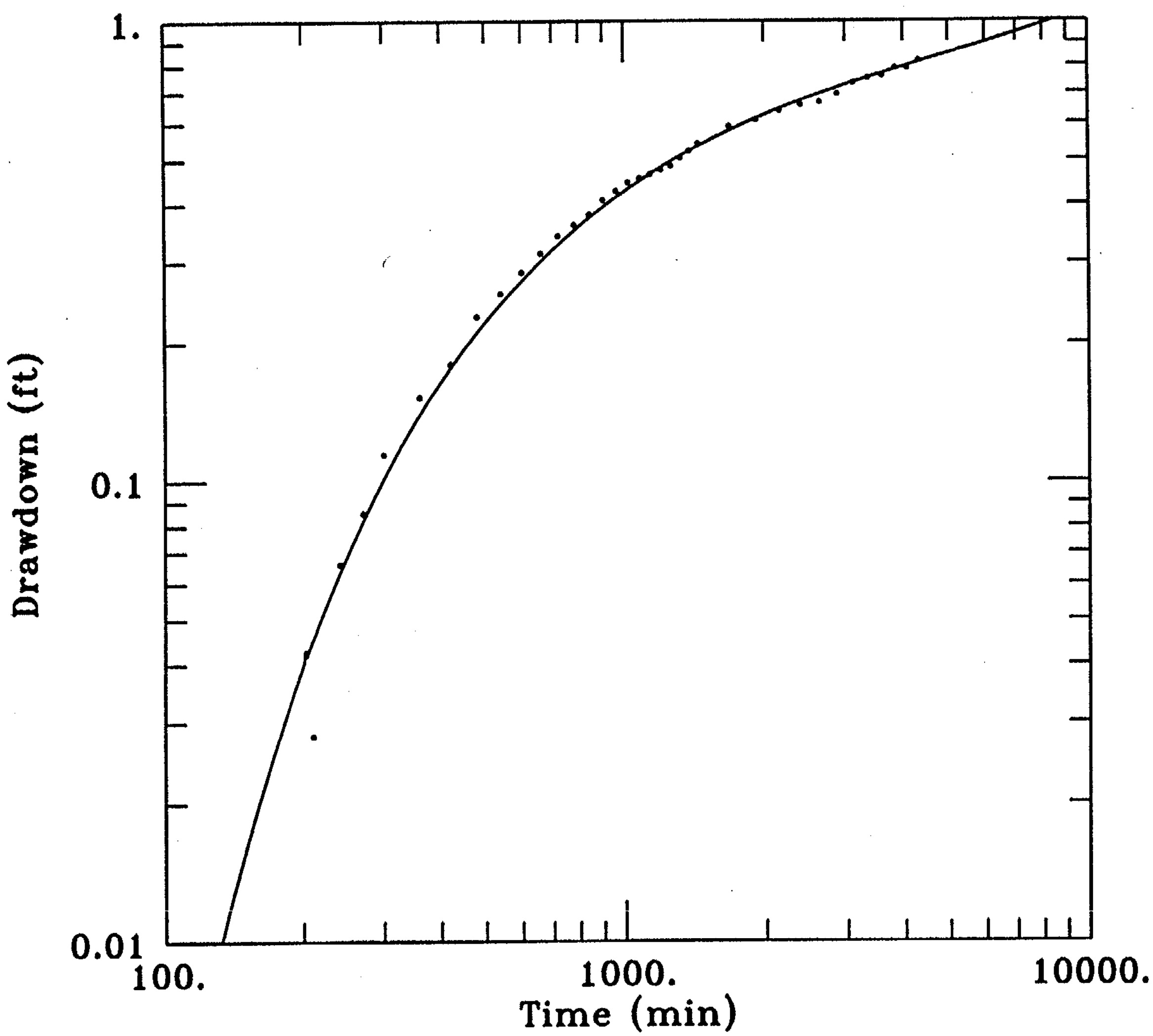
PARAMETER ESTIMATES:
T = 1231.8 ft^2/day
S' = 0.1302

AQTESOLV

Attachment H –Analytical Solution and Graphs used in the Neuman Method for the Deep Pumping Test Data

Client: **LANTDIV**	Company: **Baker Environmental, Inc.**
Location: **Site 82 -- MCB Camp Lejeune**	Project: **62470-222**

6GW-1DB DRAWDOWN DATA



DATA SET:
1DB-D.DAT
03/04/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Neuman

PROJECT DATA:
test date: February 19-24, 1996
test well: DRW-1
obs. well: 6GW-1DB

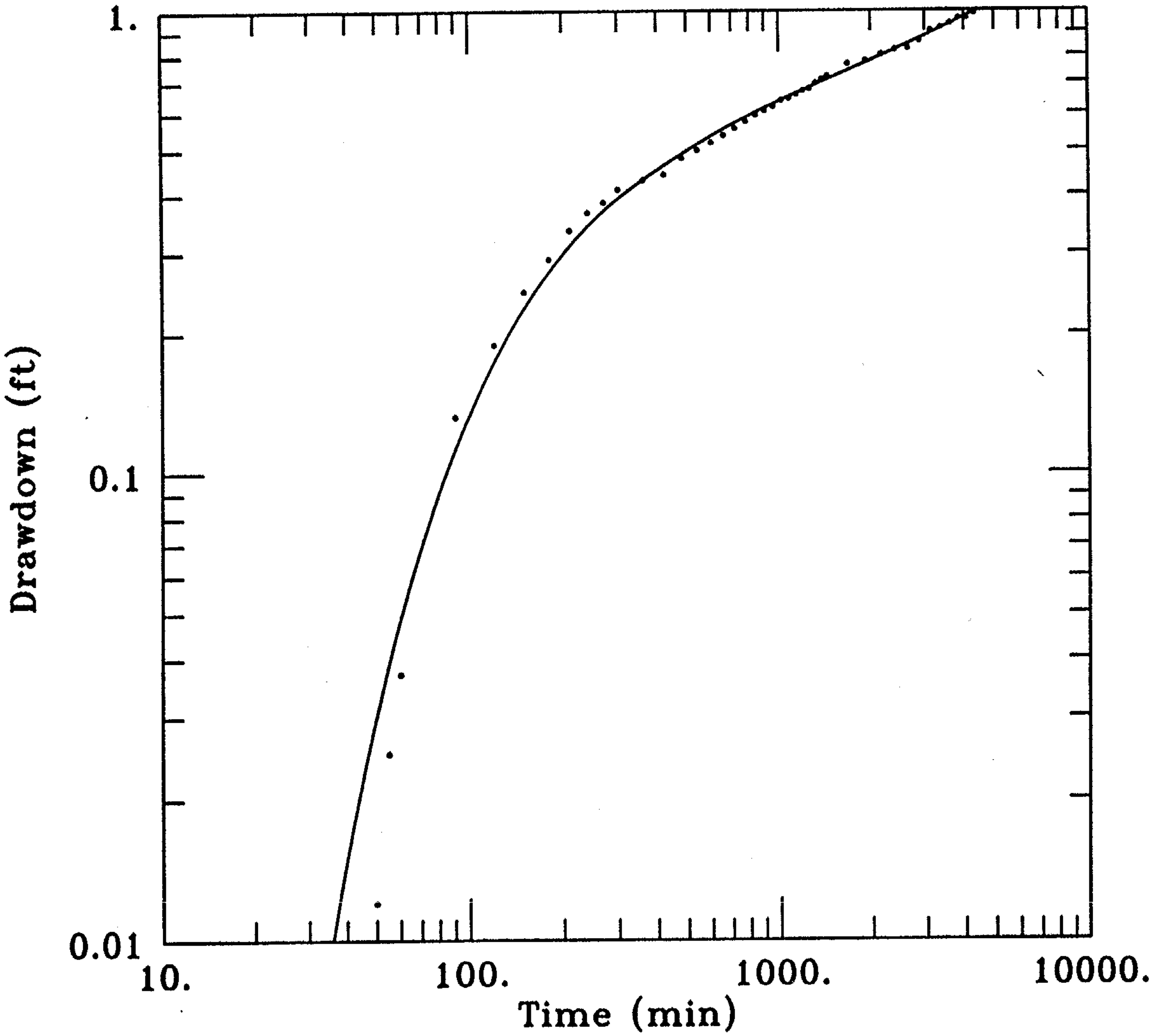
TEST DATA:
Q = 30. gal/min
r = 35.44 ft
r_c = 0.25 ft
r_w = 0.5 ft
b = 300. ft
Pumping Well Screen Depth:
top = 65. ft
bot.= 85. ft
Obs. Well Screen Depth:
top = 230. ft
bot.= 250. ft

PARAMETER ESTIMATES:
T = 556.8 ft^2/day
S = 0.08713
Sy = 0.5
β = 0.05189

AQTESOLV

Client: LANTDIV	Company: Baker Environmental, Inc.
Location: Site 82 -- MCB Camp Lejeune	Project: 62470-222

6GW-1DA DRAWDOWN DATA



DATA SET:
1DA-D.DAT
02/28/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Neuman

PROJECT DATA:
test date: February 19-24, 1996
test well: DRW-1
obs. well: 6GW-1DA

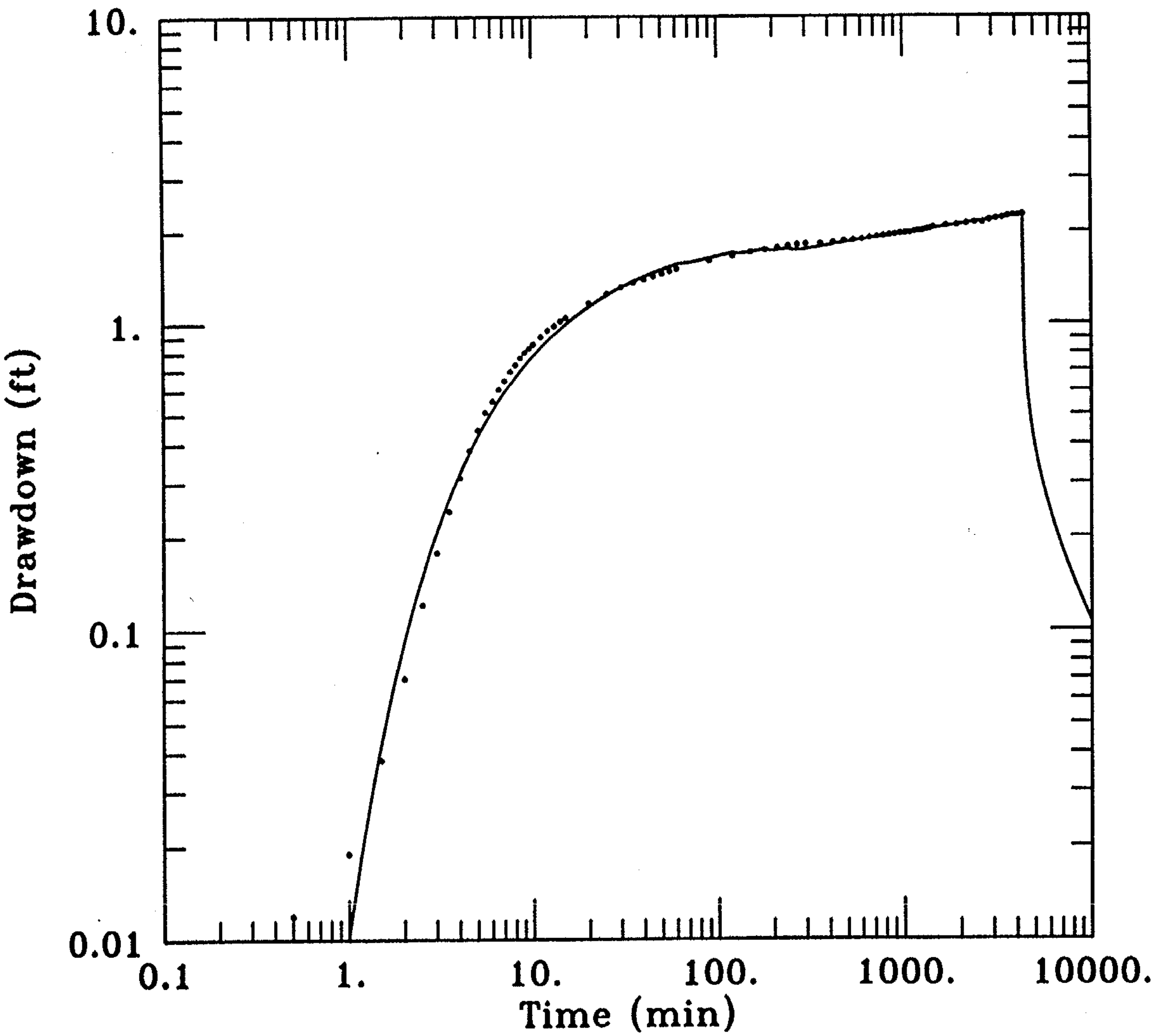
TEST DATA:
Q = 30. gal/min
r = 58.83 ft
r_c = 0.25 ft
r_w = 0.5 ft
b = 300. ft
Pumping Well Screen Depth:
top = 65. ft
bot. = 85. ft
Obs. Well Screen Depth:
top = 205. ft
bot. = 225. ft

PARAMETER ESTIMATES:
T = 783. ft^2/day
S = 0.007066
Sy = 0.03147
β = 0.02094

AQTESOLV

Client: **LANTDIV**	Company: **Baker Environmental, Inc.**
Location: **Site 82 -- MCB Camp Lejeune**	Project: **62470-222**

6GW-1D DRAWDOWN DATA



DATA SET:
1D-D.DAT
02/28/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Neuman

PROJECT DATA:
test date: February 19-24, 1996
test well: DRW-1
obs. well: 6GW-1D

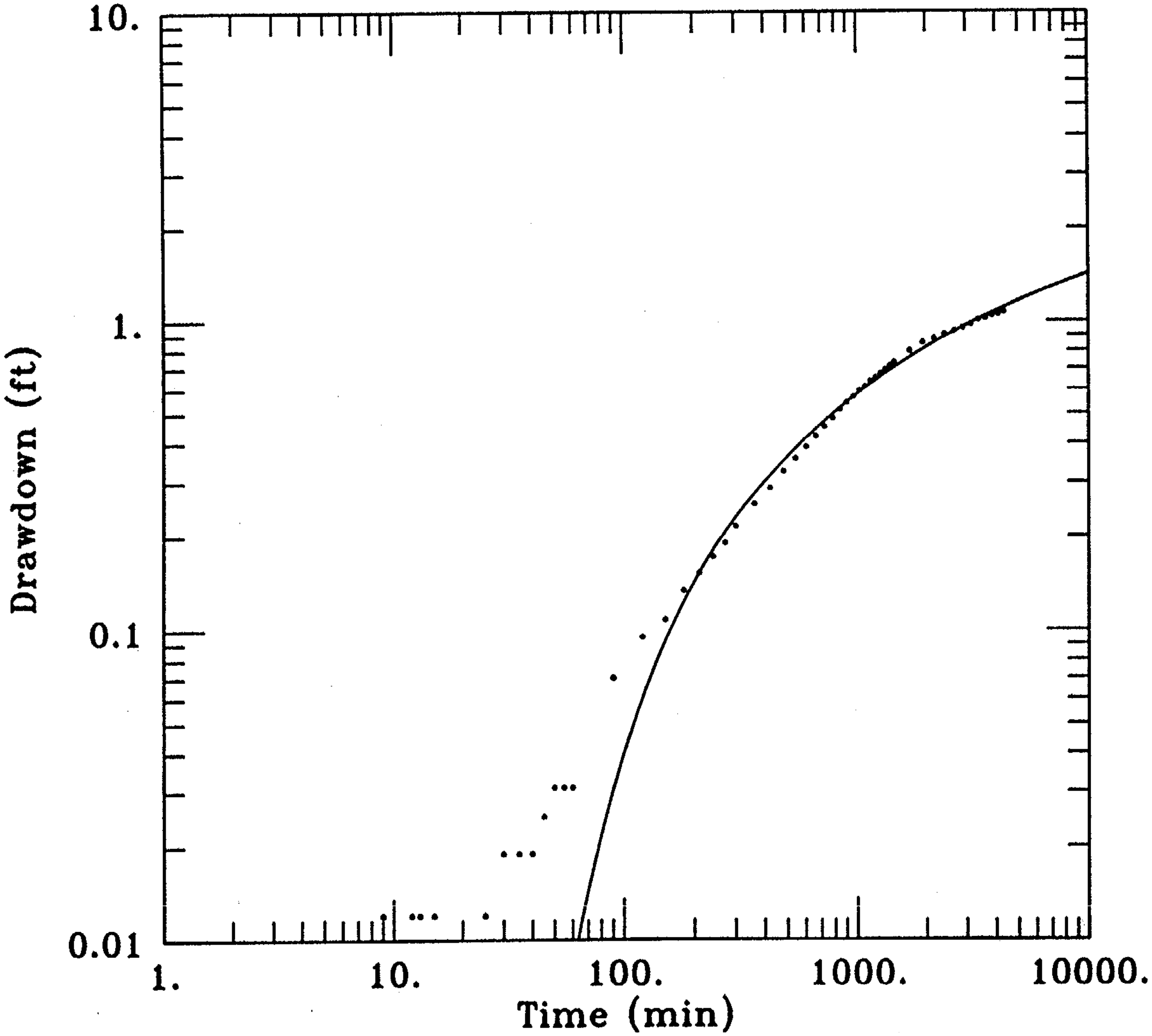
TEST DATA:
Q = 30. gal/min
r = 58.83 ft
r_c = 0.25 ft
r_w = 0.5 ft
b = 300. ft
Pumping Well Screen Depth:
top = 65. ft
bot. = 85. ft
Obs. Well Screen Depth:
top = 85. ft
bot. = 95. ft

PARAMETER ESTIMATES:
T = 2149.4 ft^2/day
S = 0.005399
Sy = 0.001
β = 0.006624

AQTESOLV

Client: LANTDIV	Company: Baker Environmental, Inc.
Location: Site 82 -- MCB Camp Lejeune	Project: 62470-222

DP-2 DRAWDOWN DATA



DATA SET:
DP-2D.DAT
03/01/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Neuman

PROJECT DATA:
test date: February 19-24, 1996
test well: DRW-1
obs. well: DP-2

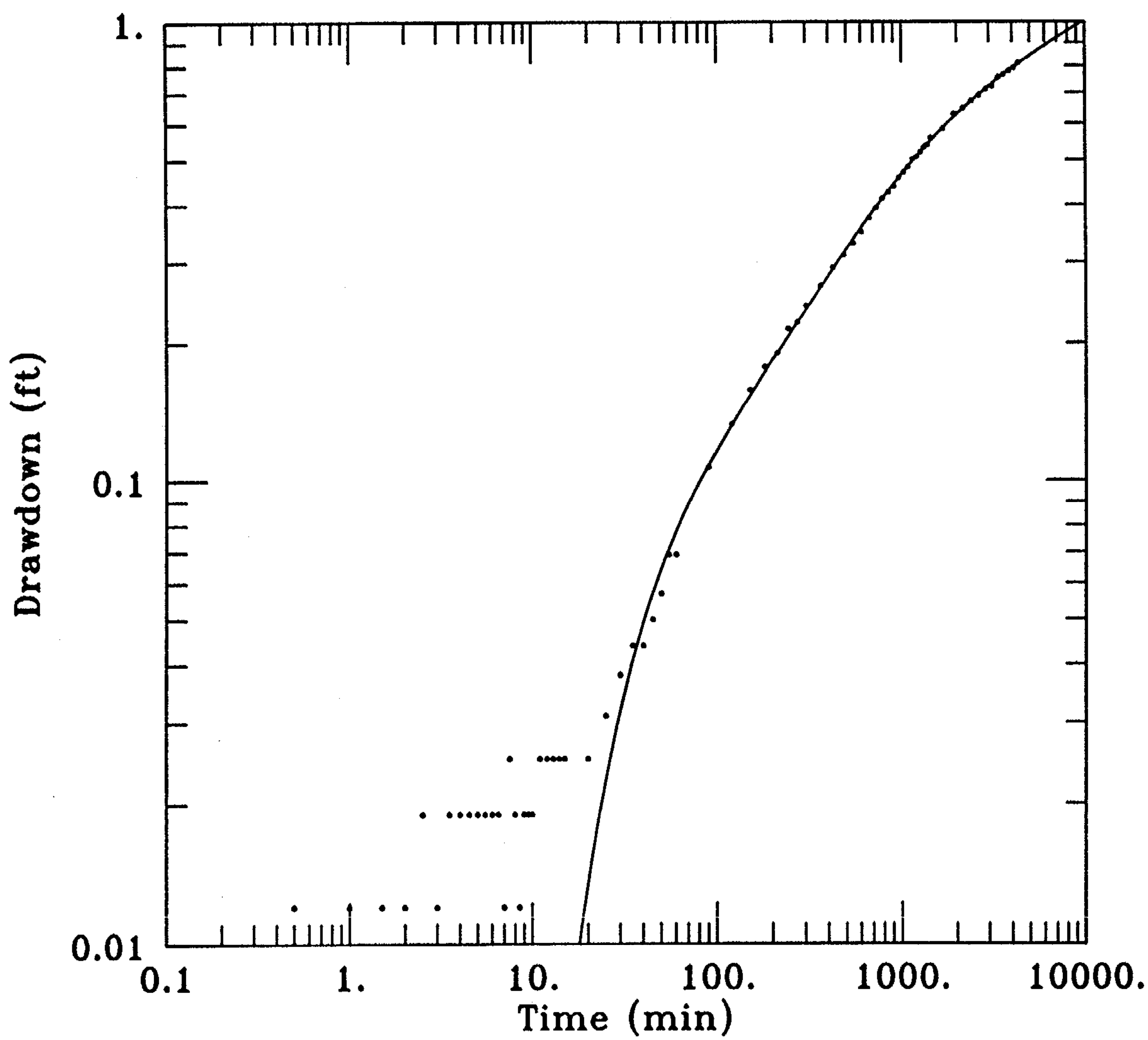
TEST DATA:
Q = 30. gal/min
r = 174.6 ft
r_c = 0.25 ft
r_w = 0.5 ft
b = 300. ft
Pumping Well Screen Depth:
top = 65. ft
bot. = 85. ft
Obs. Well Screen Depth:
top = 85. ft
bot. = 95. ft

PARAMETER ESTIMATES:
T = 1020.1 ft^2/day
S = 0.01923
Sy = 0.002046
β = 0.2007

AQTESOLV

Client: LANTDIV	Company: Baker Environmental, Inc.
Location: Site 82 -- MCB Camp Lejeune	Project: 62470-222

DP-1 DRAWDOWN DATA



DATA SET:
DP-1D.DAT
03/01/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Neuman

PROJECT DATA:
test date: February 19-24, 1996
test well: DRW-1
obs. well: DP-1

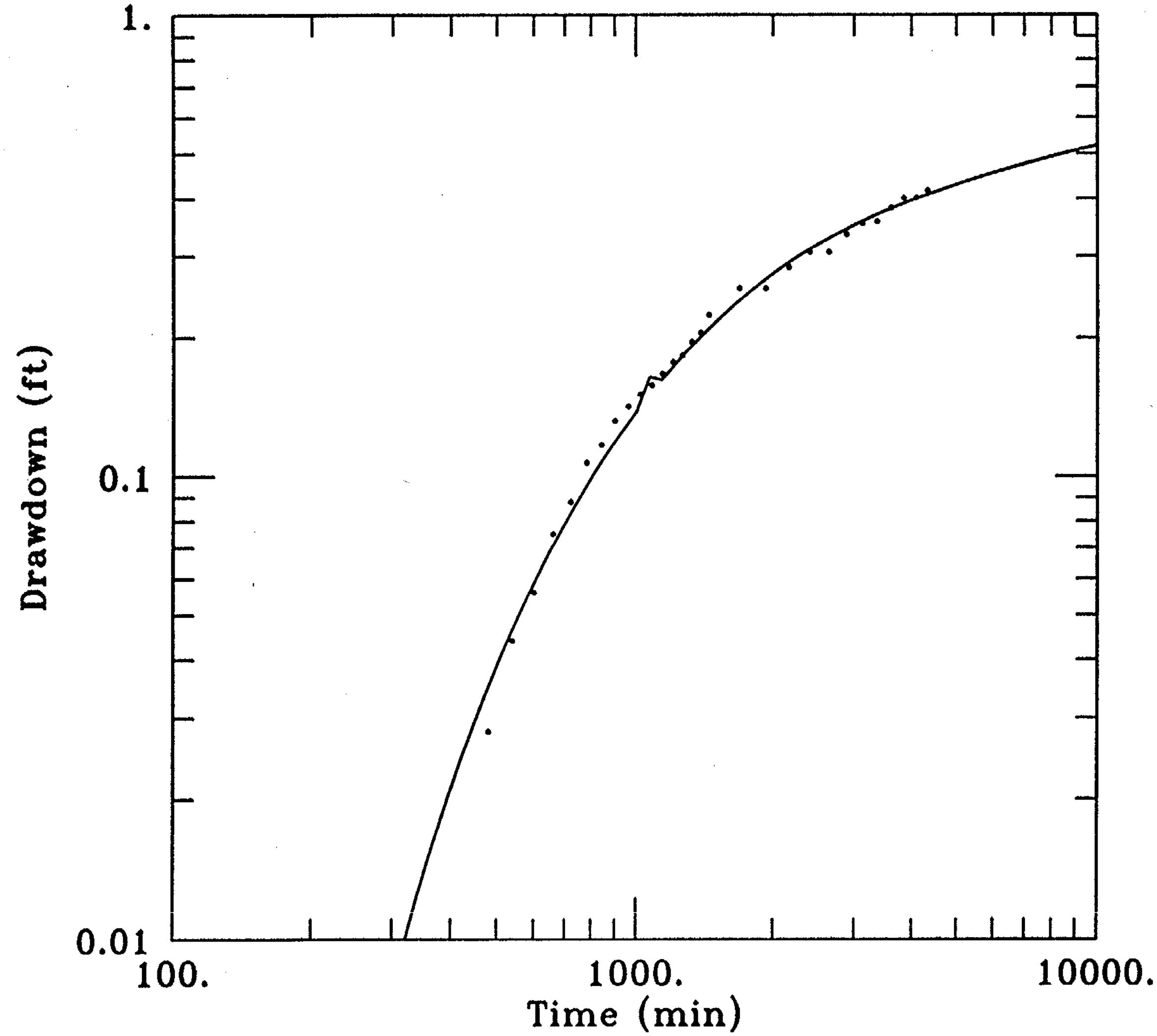
TEST DATA:
Q = 30. gal/min
r = 212.6 ft
r_c = 0.25 ft
r_w = 0.5 ft
b = 300. ft
Pumping Well Screen Depth:
top = 65. ft
bot. = 85. ft
Obs. Well Screen Depth:
top = 65. ft
bot. = 85. ft

PARAMETER ESTIMATES:
T = 1703.4 ft^2/day
S = 0.004953
Sy = 0.006026
β = 0.2988

AQTESOLV

Client: LANTDIV	Company: Baker Environmental, Inc.
Location: Site 82 -- MCB Camp Lejeune	Project: 62470-222

6GW-15D DRAWDOWN DATA



DATA SET:
15D-D.DAT
02/29/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Neuman

PROJECT DATA:
test date: February 19-24, 1996
test well: DRW-1
obs. well: 6GW-15D

TEST DATA:
Q = 30. gal/min
r = 376.5 ft
r_c = 0.25 ft
r_w = 0.5 ft
b = 300. ft
Pumping Well Screen Depth:
top = 65. ft
bot. = 85. ft
Obs. Well Screen Depth:
top = 130. ft
bot. = 140. ft

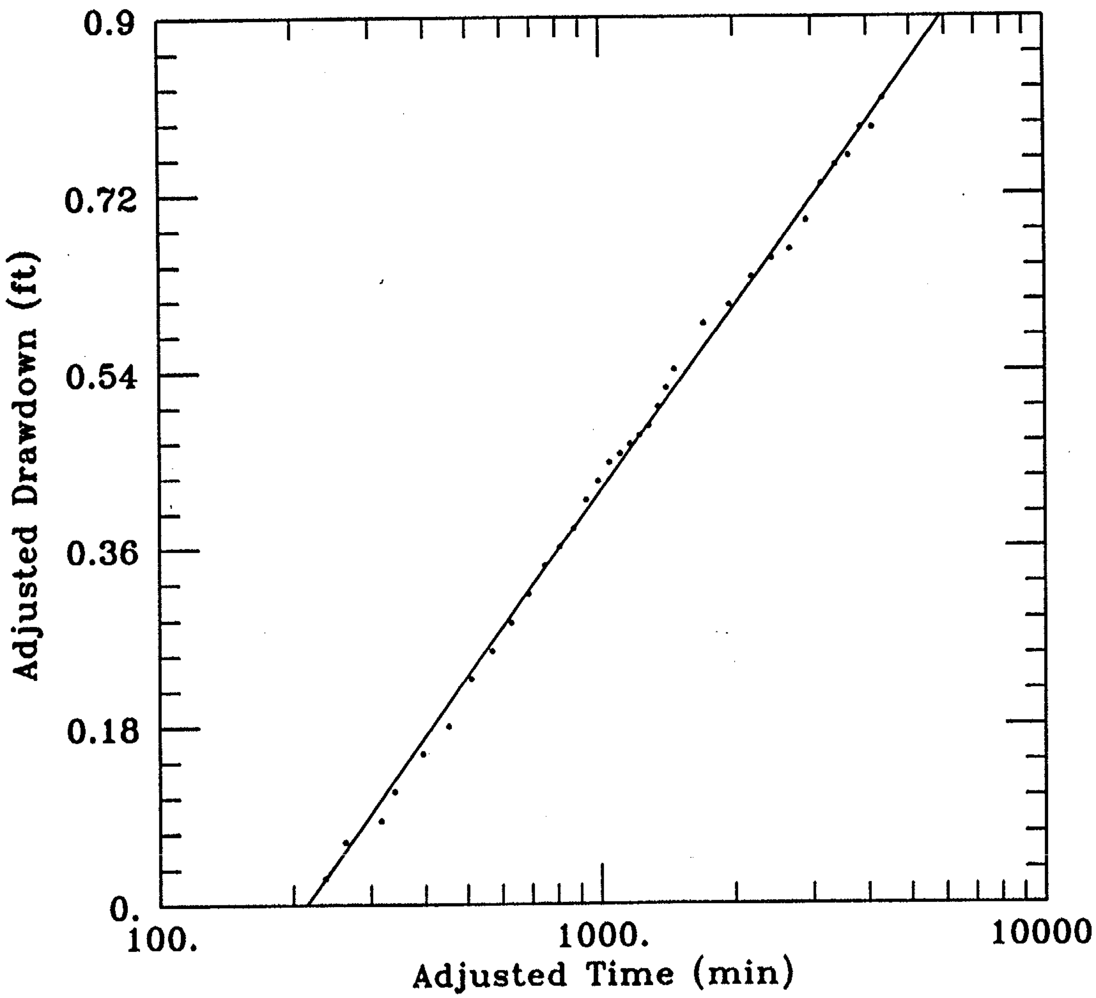
PARAMETER ESTIMATES:
T = 1133.9 ft^2/day
S = 0.01697
Sy = 0.07204
β = 0.0921

AQTESOLV

Attachment I –Analytical Solution and Graphs used in the Cooper-Jacob Method for the Deep Pumping Test Data

Client: LANTDIV	Company: Baker Environmental, Inc.
Location: Site 82 -- MCB Camp Lejeune	Project: 62470-222

6GW-1DB DRAWDOWN DATA



DATA SET:
1DB-D.DAT
02/28/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Cooper-Jacob

PROJECT DATA:
test date: February 19-24, 1996
test well: DRW-1
obs. well: 6GW-1DB

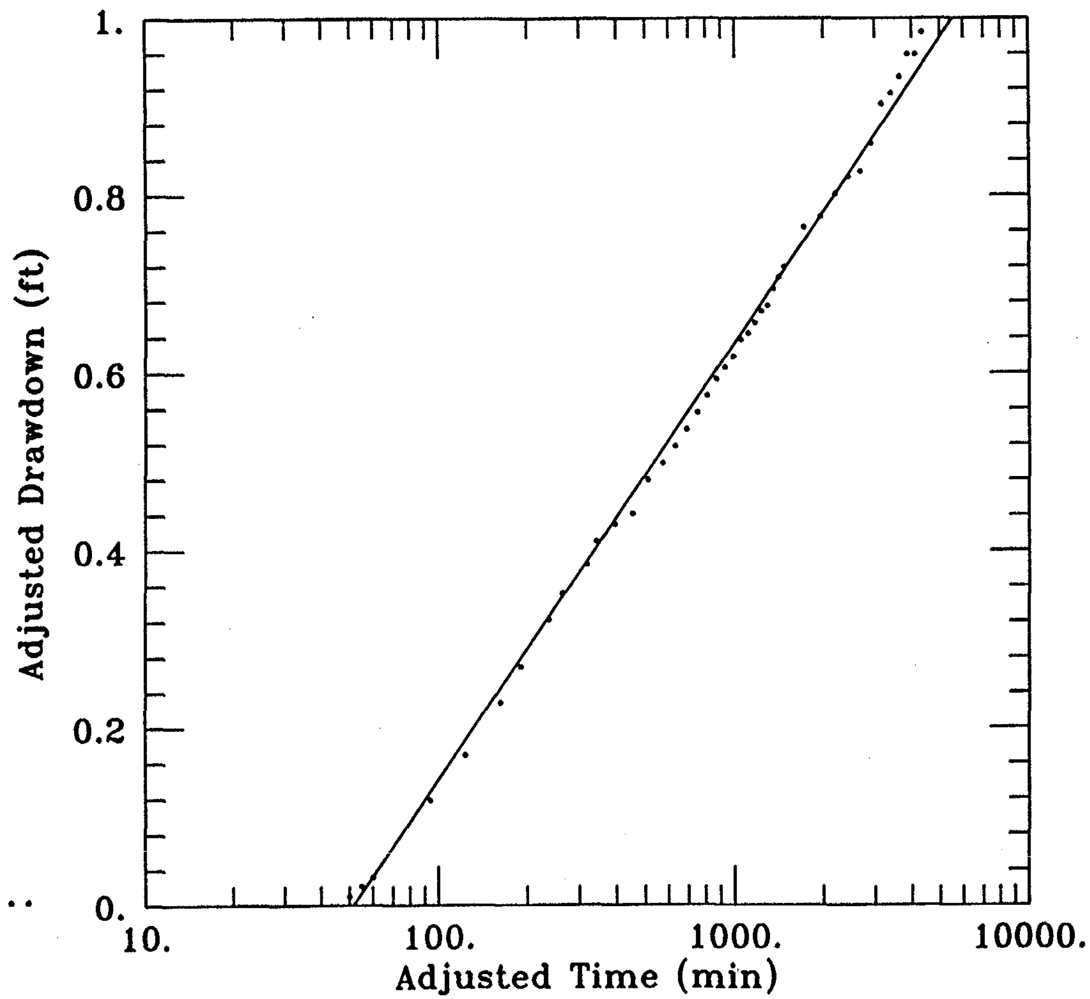
TEST DATA:
Q = 30. gal/min
r = 35.44 ft
r_c = 0.25 ft
r_w = 0.5 ft
b = 300. ft

PARAMETER ESTIMATES:
T = 1466.1 ft^2/day
S = 0.3909

AQTESOLV

Client: LANTDIV	Company: Baker Environmental, Inc.
Location: Site 82 -- MCB Camp Lejeune	Project: 62470-222

6GW-1DA DRAWDOWN DATA



DATA SET:
1DA-D.DAT
02/28/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Cooper-Jacob

PROJECT DATA:
test date: February 19-24, 1996
test well: DRW-1
obs. well: 6GW-1DA

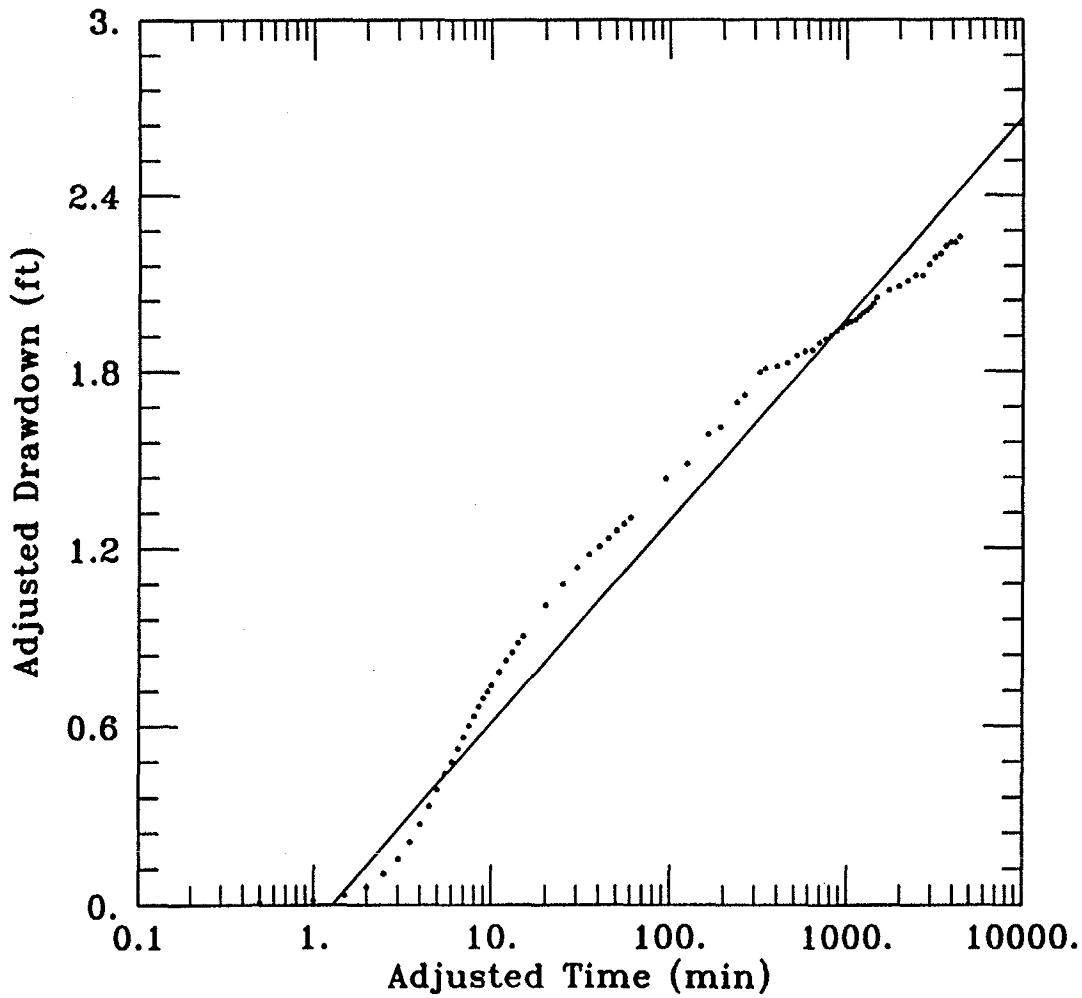
TEST DATA:
Q = 30. gal/min
r = 58.83 ft
r_c = 0.25 ft
r_w = 0.5 ft
b = 300. ft

PARAMETER ESTIMATES:
T = 1863.2 ft^2/day
S = 0.04327

AQTESOLV

Client: **LANTDIV**	Company: **Baker Environmental, Inc.**
Location: **Site 82 -- MCB Camp Lejeune**	Project: **62470-222**

6GW-1D DRAWDOWN DATA



DATA SET:
1D-D.DAT
02/28/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Cooper-Jacob

PROJECT DATA:
test date: February 19-24, 1996
test well: DRW-1
obs. well: 6GW-1D

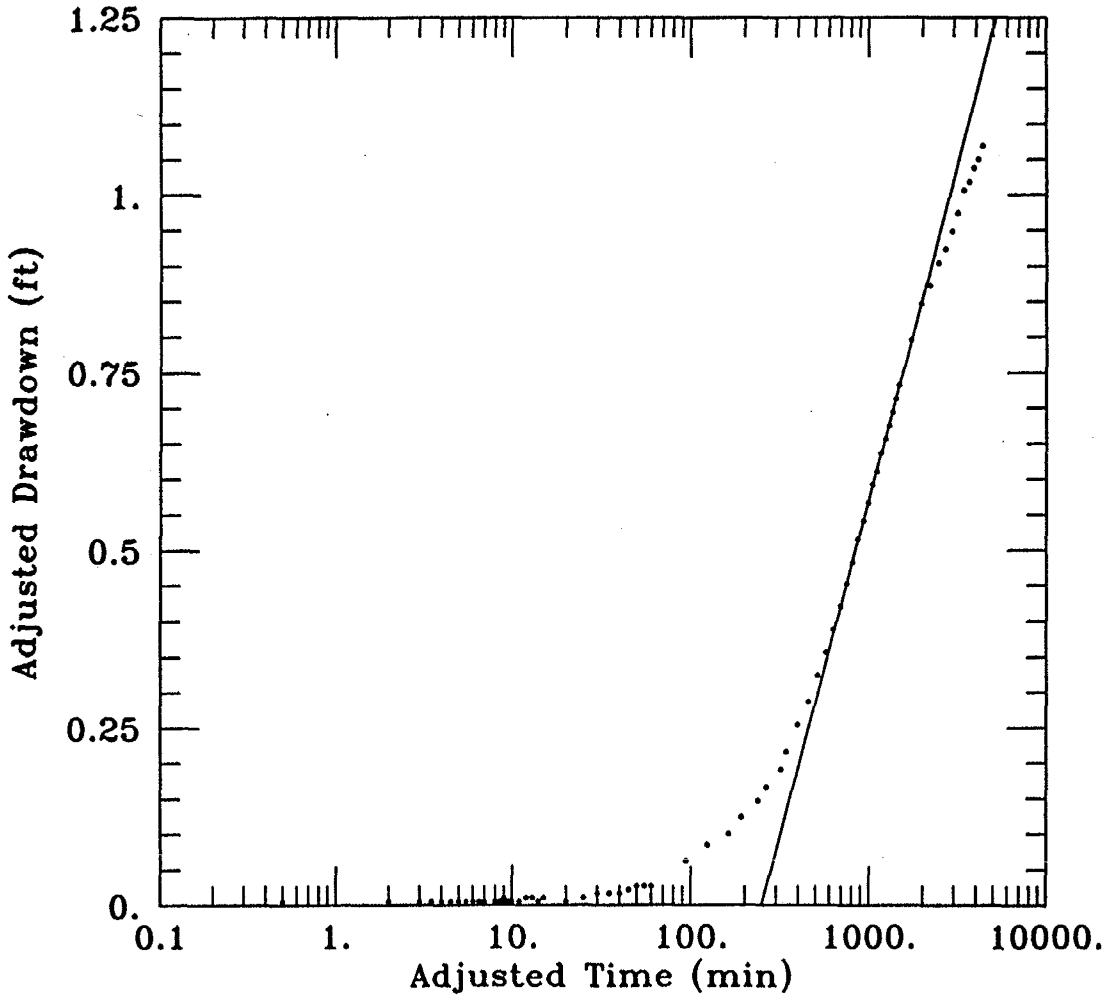
TEST DATA:
Q = 30. gal/min
r = 58.83 ft
r_c= 0.25 ft
r_w= 0.5 ft
b = 300. ft

PARAMETER ESTIMATES:
T = 1339.2 ft^2/day
S = 0.0007713

AQTESOLV

Client: **LANTDIV**	Company: **Baker Environmental, Inc.**
Location: **Site 82 -- MCB Camp Lejeune**	Project: **62470-222**

DP-2 DRAWDOWN DATA



DATA SET:
DP-2DE.DAT
03/01/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Cooper-Jacob

PROJECT DATA:
test date: February 19-24, 1996
test well: DRW-1
obs. well: DP-2

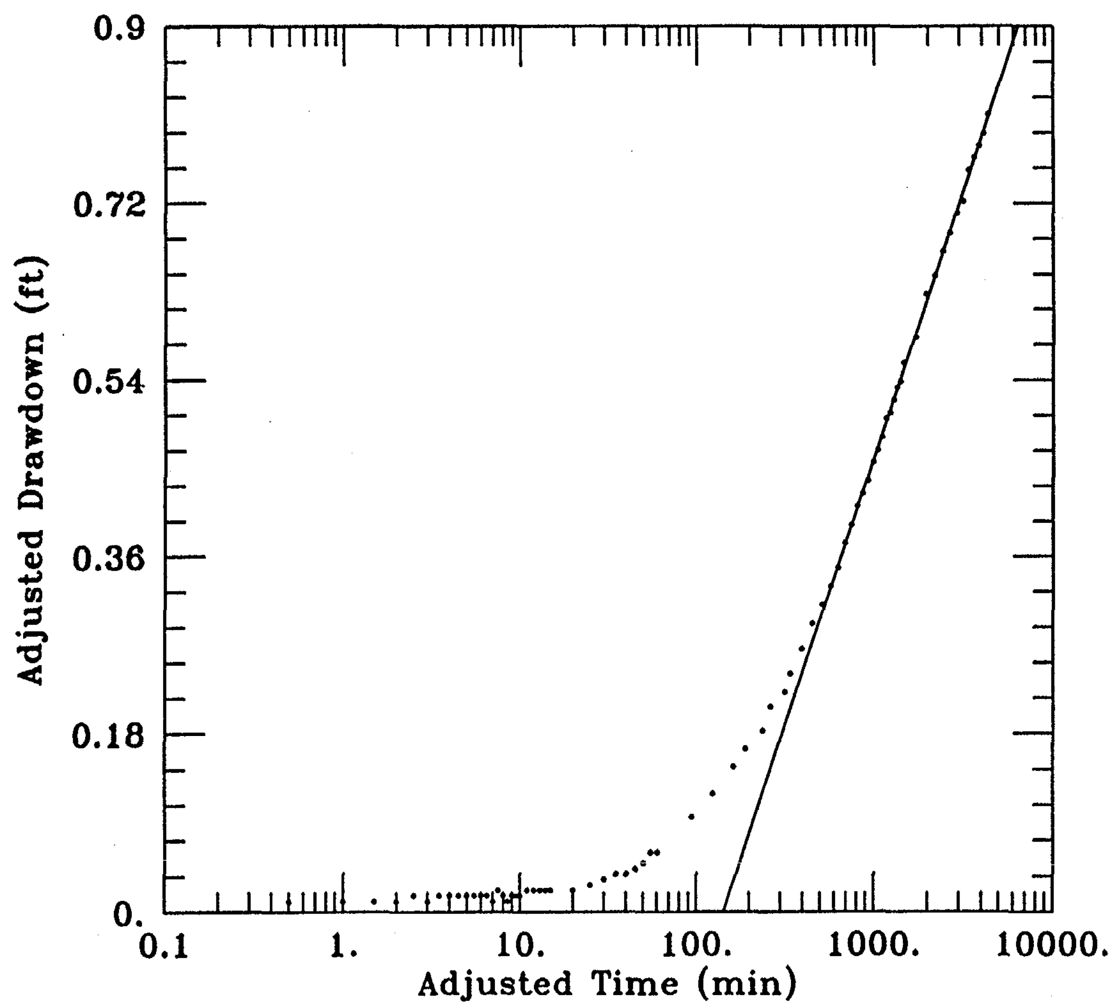
TEST DATA:
Q = 30. gal/min
r = 174.6 ft
r_c = 0.25 ft
r_w = 0.5 ft
b = 300. ft

PARAMETER ESTIMATES:
T = 969.1 ft^2/day
S = 0.0123

AQTESOLV

Client: LANTDIV	Company: Baker Environmental, Inc.
Location: Site 82 -- MCB Camp Lejeune	Project: 62470-222

DP-1 DRAWDOWN DATA



DATA SET:
DP-1D.DAT
02/28/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Cooper-Jacob

PROJECT DATA:
test date: February 19-24, 1996
test well: DRW-1
obs. well: DP-1

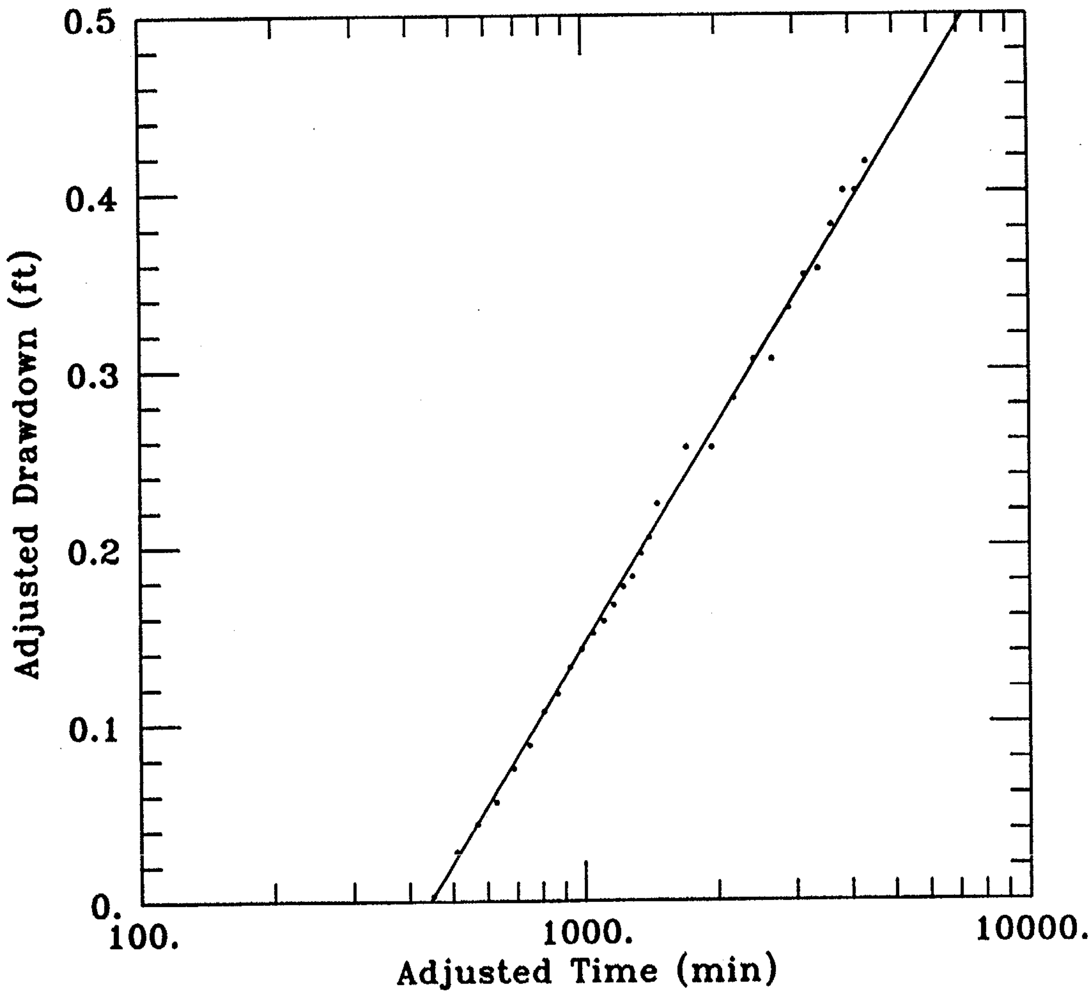
TEST DATA:
Q = 30. gal/min
r = 212.6 ft
r_c = 0.25 ft
r_w = 0.5 ft
b = 300. ft

PARAMETER ESTIMATES:
T = 1684.5 ft^2/day
S = 0.008291

AQTESOLV

Client: **LANTDIV**	Company: **Baker Environmental, Inc.**
Location: **Site 82 -- MCB Camp Lejeune**	Project: **62470-222**

6GW-15D DRAWDOWN DATA



DATA SET:
15D-D.DAT
02/28/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Cooper-Jacob

PROJECT DATA:
test date: February 19-24, 1996
test well: DRW-1
obs. well: 6GW-15D

TEST DATA:
Q = 30. gal/min
r = 376.5 ft
r_c = 0.25 ft
r_w = 0.5 ft
b = 300. ft

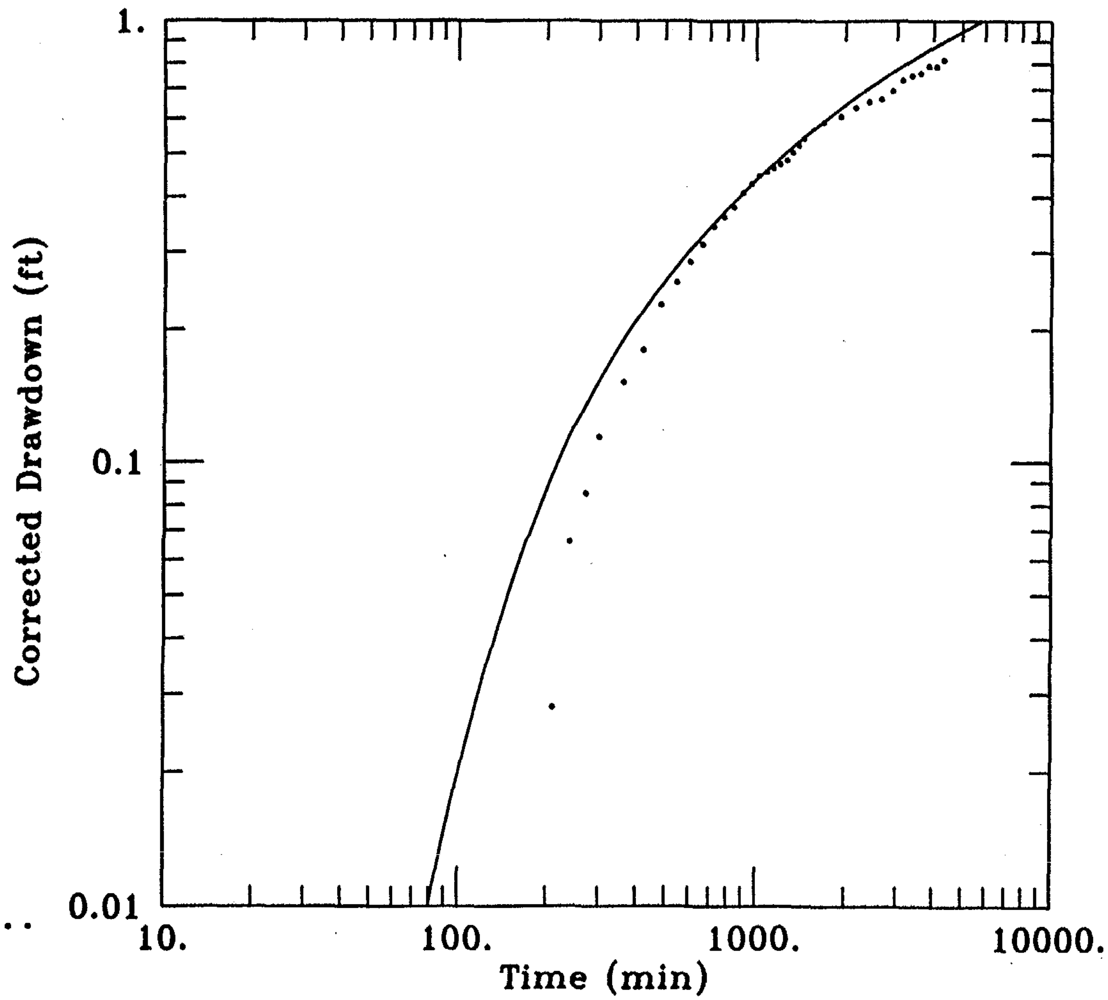
PARAMETER ESTIMATES:
T = 2221.9 ft^2/day
S = 0.01089

AQTESOLV

Attachment J – Analytical Solution and Graphs used in the Theis Method for the Deep Pumping Test Data

Client: **LANTDIV**	Company: **Baker Environmental, Inc.**
Location: **Site 82 -- MCB Camp Lejeune**	Project: **62470-222**

6GW-1DB DRAWDOWN DATA



DATA SET:
1DB-D.DAT
02/28/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Theis

PROJECT DATA:
test date: February 19-24, 1996
test well: DRW-1
obs. well: 6GW-1DB

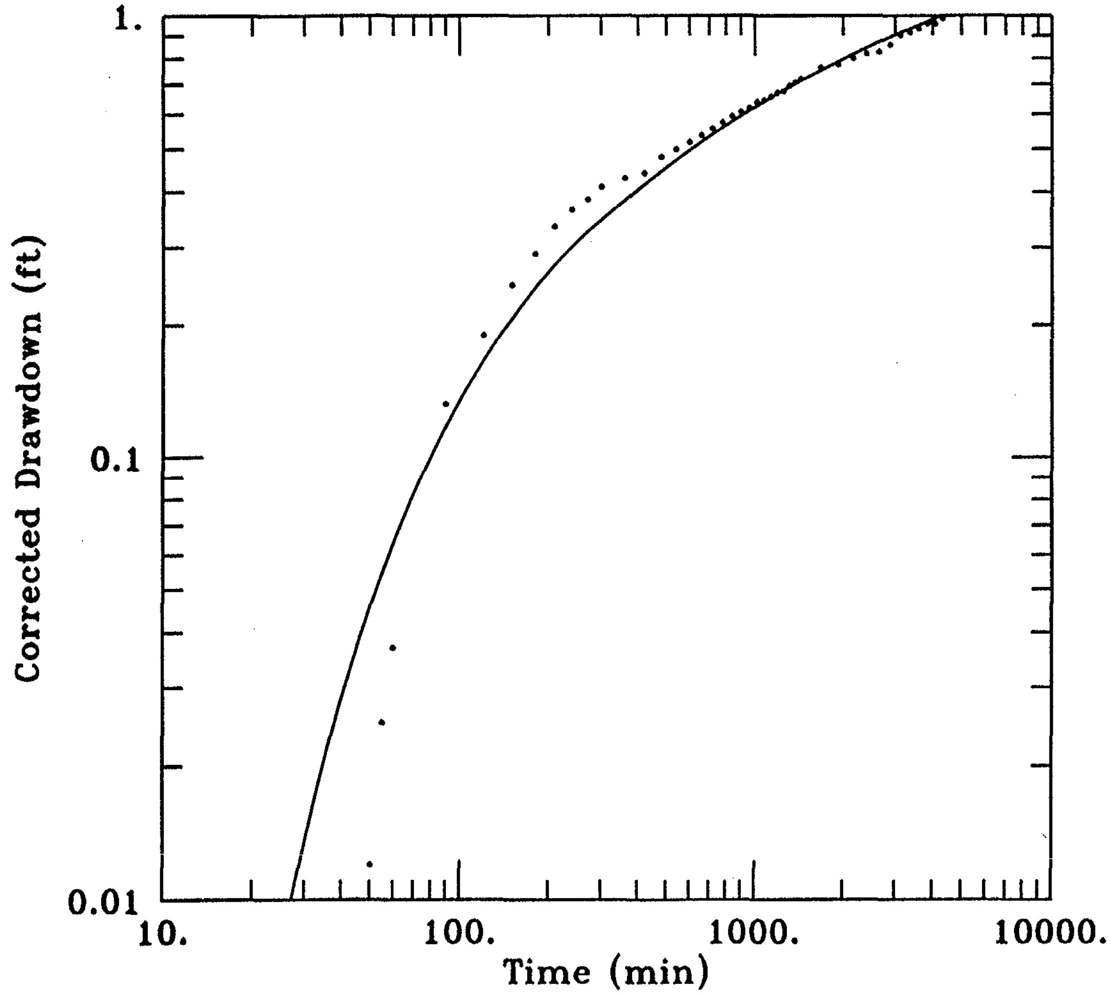
TEST DATA:
Q = 30. gal/min
r = 35.44 ft
r_c = 0.25 ft
r_w = 0.5 ft
b = 300. ft

PARAMETER ESTIMATES:
T = 1130.8 ft^2/day
S = 0.5

AQTESOLV

Client: LANTDIV	Company: Baker Environmental, Inc.
Location: Site 82 -- MCB Camp Lejeune	Project: 62470-222

6GW-1DA DRAWDOWN DATA



DATA SET:
1DA-D.DAT
02/28/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Theis

PROJECT DATA:
test date: February 19-24, 1996
test well: DRW-1
obs. well: 6GW-1DA

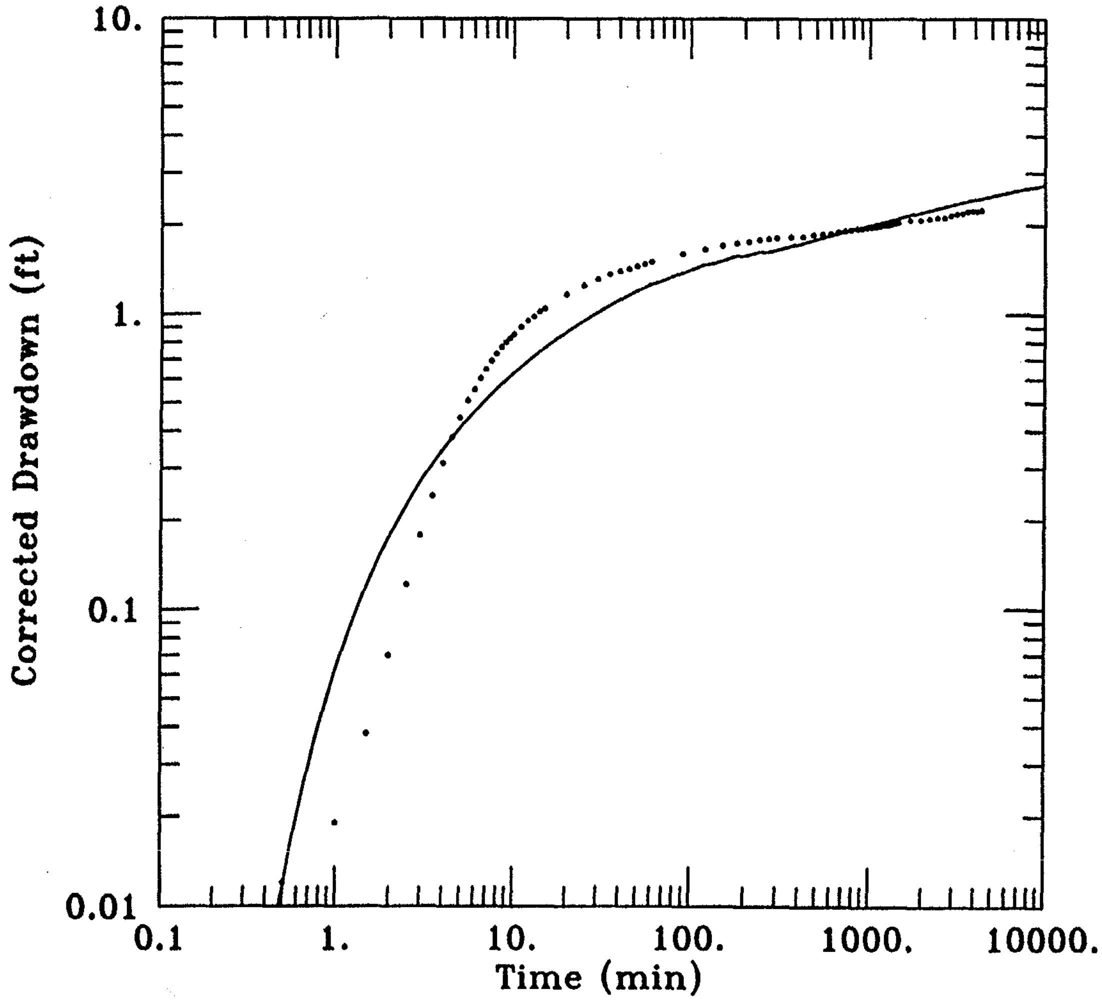
TEST DATA:
Q = 30. gal/min
r = 58.83 ft
r_c = 0.25 ft
r_w = 0.5 ft
b = 300. ft

PARAMETER ESTIMATES:
T = 1461.7 ft^2/day
S = 0.07369

AQTESOLV

Client: LANTDIV	Company: Baker Environmental, Inc.
Location: Site 82 -- MCB Camp Lejeune	Project: 62470-222

6GW-1D DRAWDOWN DATA



DATA SET:
1D-D.DAT
02/28/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Theis

PROJECT DATA:
test date: February 19-24, 1996
test well: DRW-1
obs. well: 6GW-1D

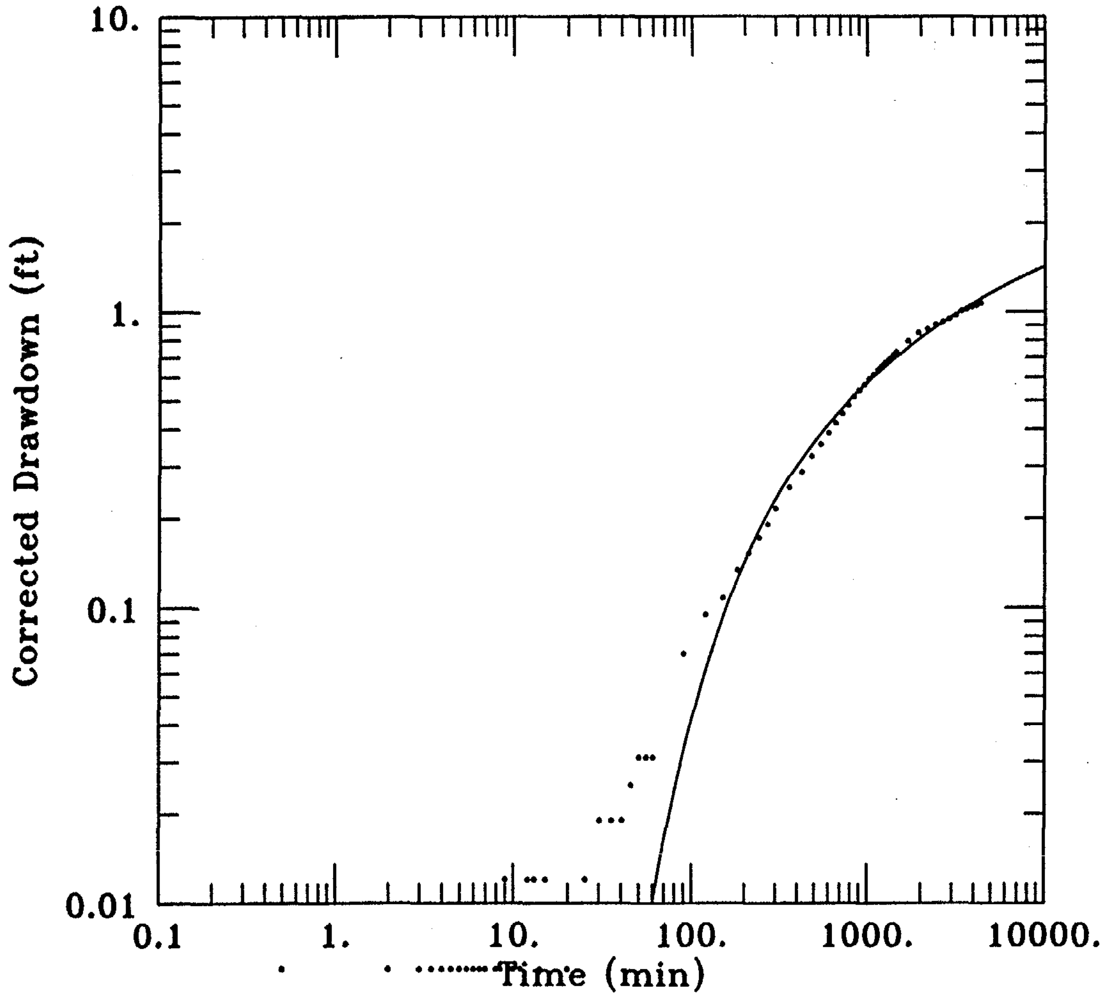
TEST DATA:
Q = 30. gal/min
r = 58.83 ft
r_c = 0.25 ft
r_w = 0.5 ft
b = 300. ft

PARAMETER ESTIMATES:
T = 1234. ft^2/day
S = 0.001149

AQTESOLV

Client: LANTDIV	Company: Baker Environmental, Inc.
Location: Site 82 -- MCB Camp Lejeune	Project: 62470-222

DP-2 DRAWDOWN DATA



DATA SET:
DP-2D.DAT
03/01/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Theis

PROJECT DATA:
test date: February 19-24, 1996
test well: DRW-1
obs. well: DP-2

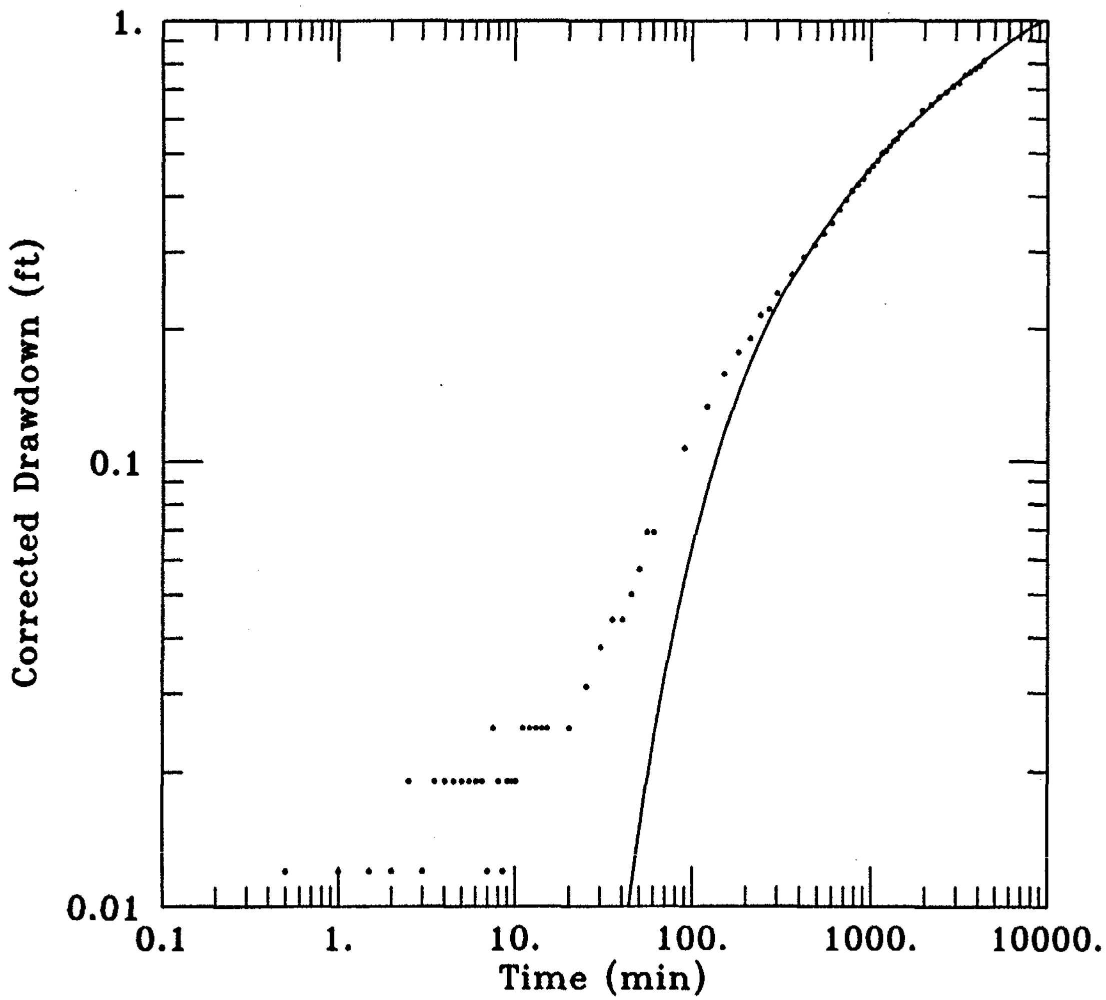
TEST DATA:
Q = 30. gal/min
r = 174.6 ft
r_c = 0.25 ft
r_w = 0.5 ft
b = 300. ft

PARAMETER ESTIMATES:
T = 1005.1 ft^2/day
S = 0.01424

AQTESOLV

Client: **LANTDIV**	Company: **Baker Environmental, Inc.**
Location: **Site 82 -- MCB Camp Lejeune**	Project: **62470-222**

DP-1 DRAWDOWN DATA



DATA SET:
DP-1D.DAT
02/29/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Theis

PROJECT DATA:
test date: February 19-24, 1996
test well: DRW-1
obs. well: DP-1

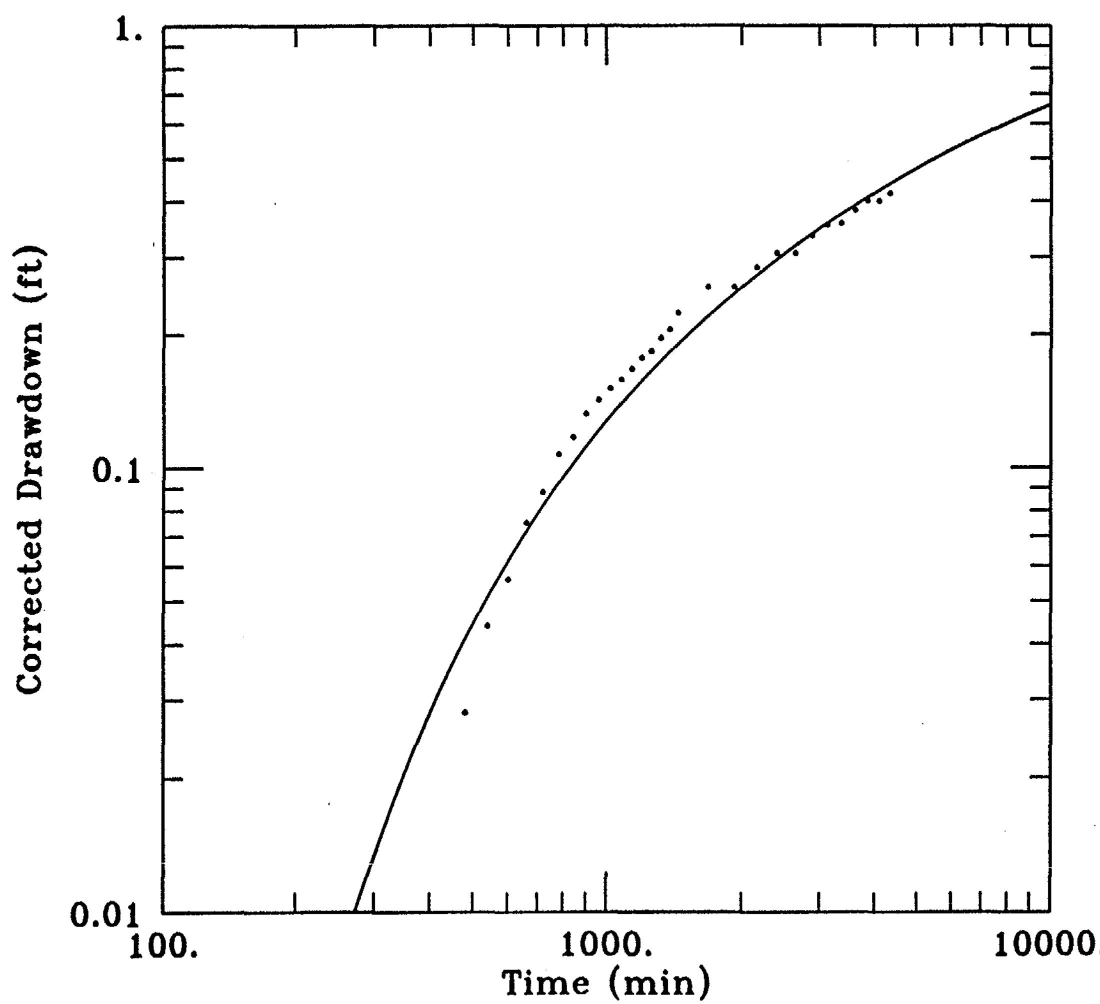
TEST DATA:
Q = 30. gal/min
r = 212.6 ft
r_c = 0.25 ft
r_w = 0.5 ft
b = 300. ft

PARAMETER ESTIMATES:
T = 1587.4 ft^2/day
S = 0.009623

AQTESOLV

Client: LANTDIV	Company: Baker Environmental, Inc.
Location: Site 82 -- MCB Camp Lejeune	Project: 62470-222

6GW-15D DRAWDOWN DATA



DATA SET:
15D-D.DAT
02/28/96

AQUIFER MODEL:
Unconfined

SOLUTION METHOD:
Theis

PROJECT DATA:
test date: February 19-24, 1996
test well: DRW-1
obs. well: 6GW-15D

TEST DATA:
Q = 30. gal/min
r = 376.5 ft
r_c = 0.25 ft
r_w = 0.5 ft
b = 300. ft

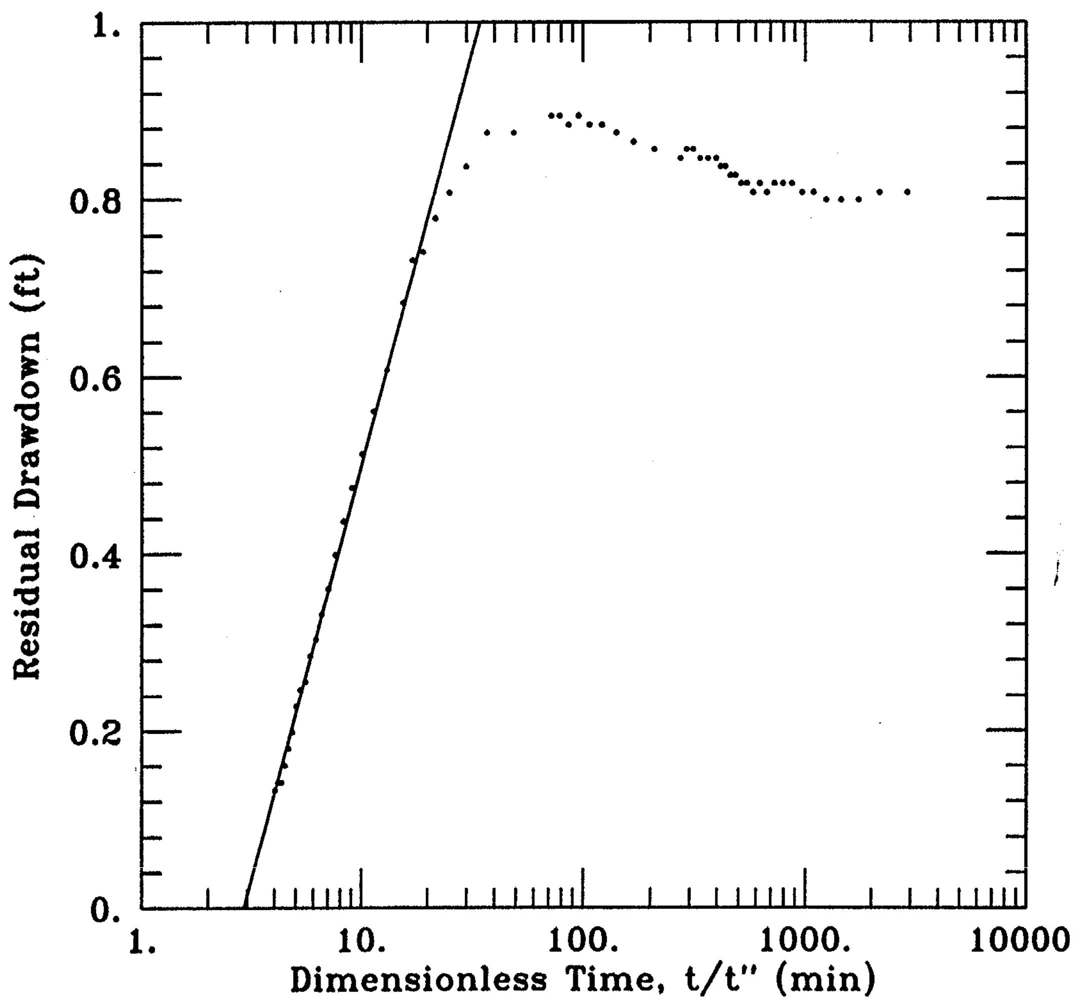
PARAMETER ESTIMATES:
T = 1344.7 ft^2/day
S = 0.01693

AQTESOLV

Attachment K – Analytical Solution and Graphs used in the Theis Recovery Method

Client: LANTDIV	Company: Baker Environmental, Inc.
Location: Site 82 -- MCB Camp Lejeune	Project: 62470-222

6GW-1DB RECOVERY DATA



DATA SET:
1DB-R.DAT
02/26/96

AQUIFER MODEL:
Confined

SOLUTION METHOD:
Theis Recovery

PROJECT DATA:
test date: February 19-24, 1996
test well: DRW-1
obs. well: 6GW-1DB

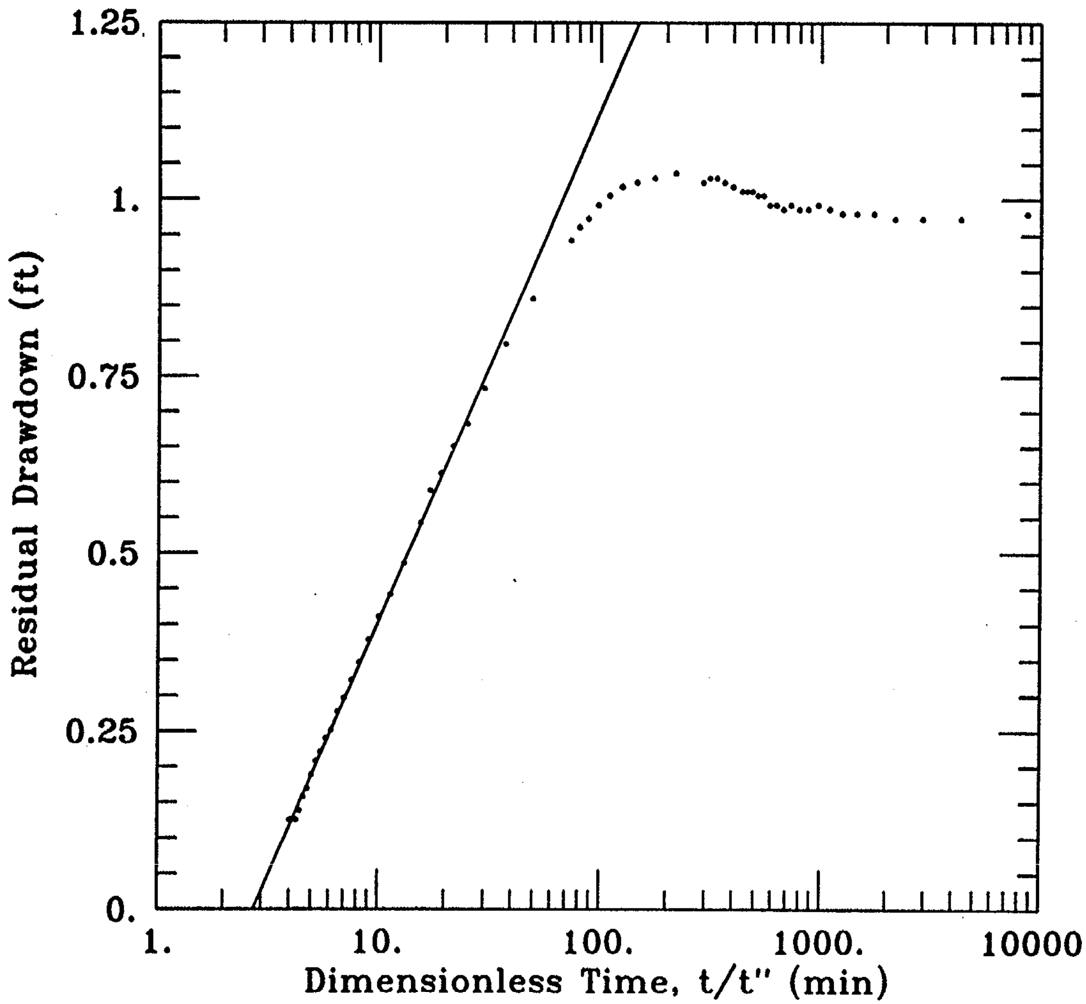
TEST DATA:
Q = 30. gal/min
r = 37.94 ft
r_c = 0.25 ft
r_w = 0.5 ft
b = 300. ft

PARAMETER ESTIMATES:
T = 986.9 ft^2/day
S' = 2.892

AQTESOLV

Client: **LANTDIV**	Company: **Baker Environmental, Inc.**
Location: **Site 82 -- MCB Camp Lejeune**	Project: **62470-222**

6GW-1DA RECOVERY DATA



DATA SET:
1DA-R.DAT
02/26/96

AQUIFER MODEL:
Confined

SOLUTION METHOD:
Theis Recovery

PROJECT DATA:
test date: February 19-24, 1996
test well: DRW-1
obs. well: 6GW-1DA

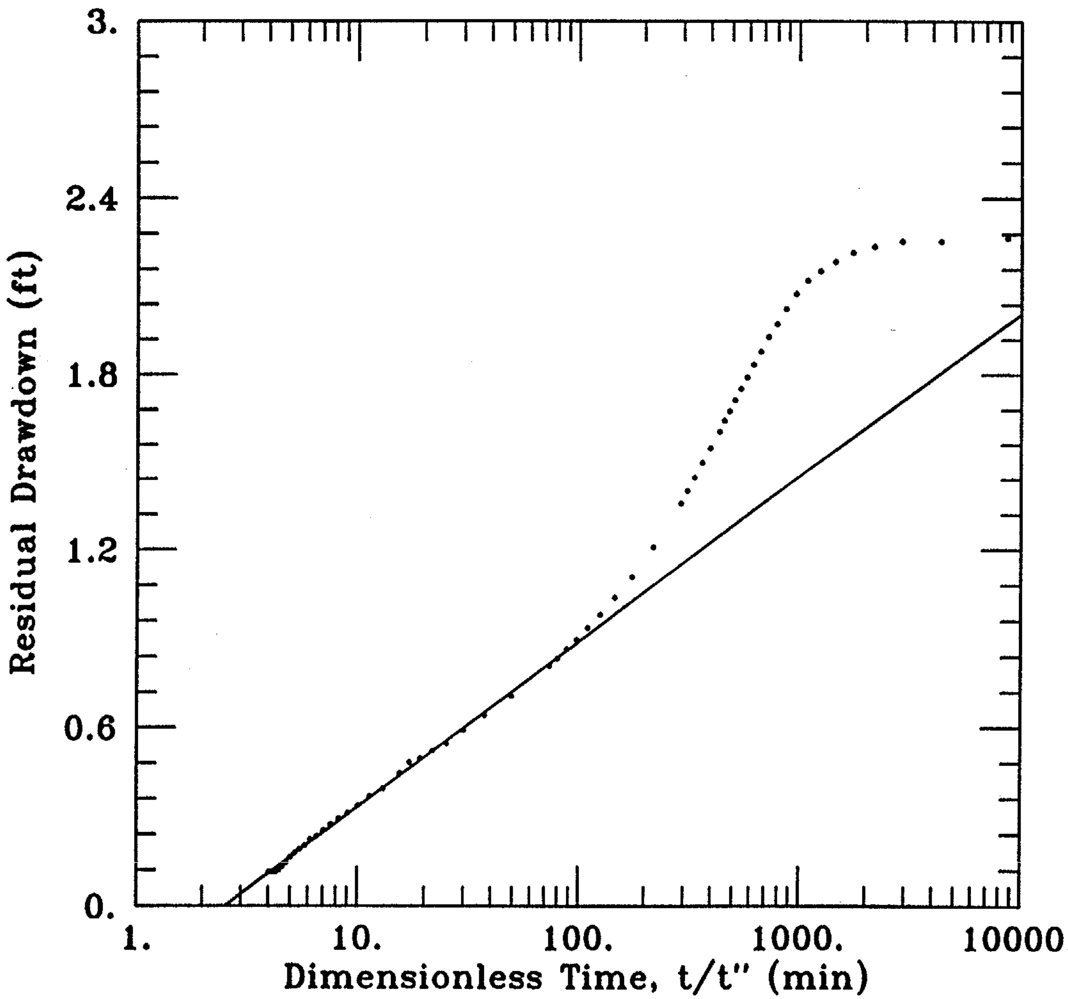
TEST DATA:
Q = 30. gal/min
r = 41.93 ft
r_c = 0.25 ft
r_w = 0.5 ft
b = 300. ft

PARAMETER ESTIMATES:
T = 1270.8 ft^2/day
S' = 2.759

AQTESOLV

Client: LANTDIV	Company: Baker Environmental, Inc.
Location: Site 82 -- MCB Camp Lejeune	Project: 62470-222

6GW-1D RECOVERY DATA



DATA SET:
1D-LR.DAT
02/26/96

AQUIFER MODEL:
Confined

SOLUTION METHOD:
Theis Recovery

PROJECT DATA:
test date: February 19-24, 1996
test well: DRW-1
obs. well: 6GW-1D

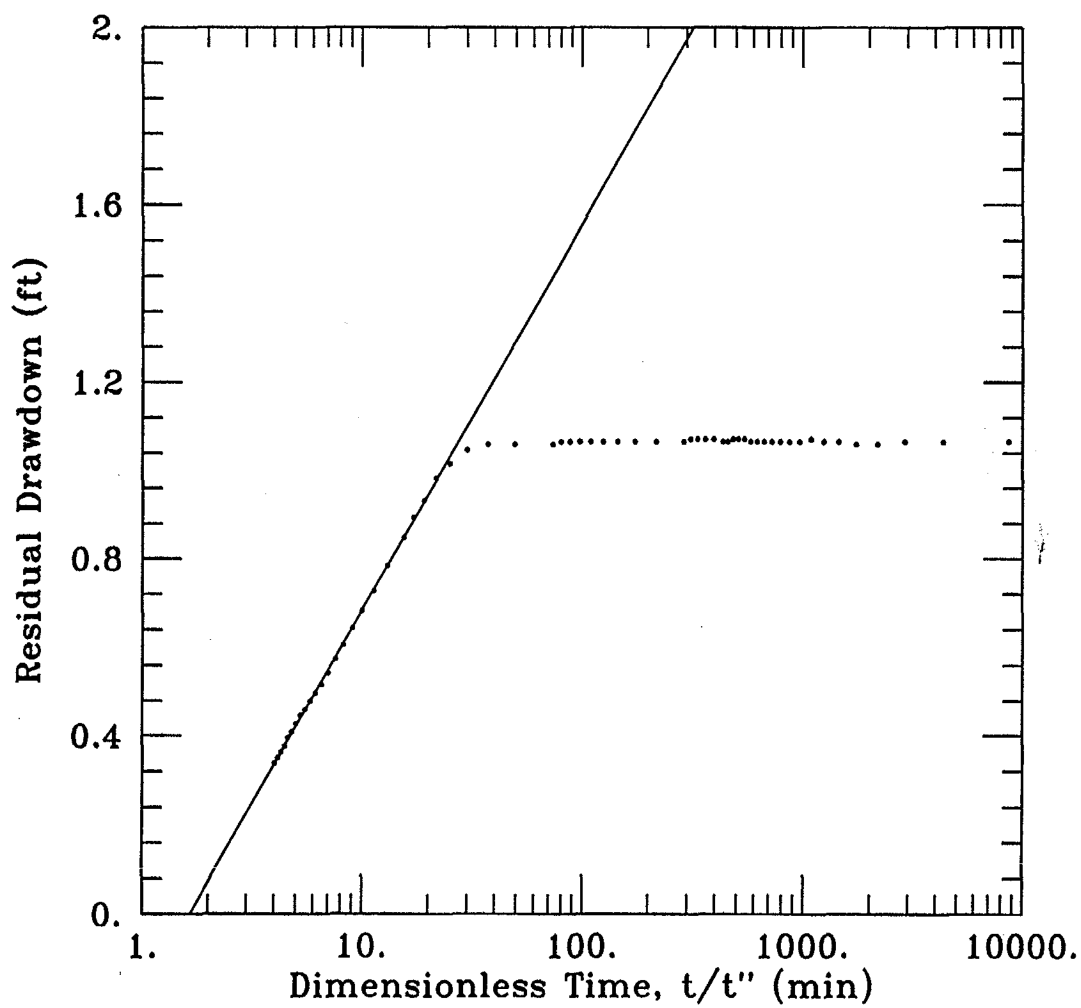
TEST DATA:
Q = 30. gal/min
r = 56.39 ft
r_c = 0.25 ft
r_w = 0.5 ft
b = 300. ft

PARAMETER ESTIMATES:
T = 1641.5 ft^2/day
S' = 2.54

AQTESOLV

Client: LANTDIV	Company: Baker Environmental, Inc.
Location: Site 82 -- MCB Camp Lejeune	Project: 62470-222

DP-2 RECOVERY DATA



DATA SET:
DP-2R.DAT
07/12/96

AQUIFER MODEL:
Confined

SOLUTION METHOD:
Theis Recovery

PROJECT DATA:
test date: February 19-24, 1996
test well: DRW-1
obs. well: DP-2

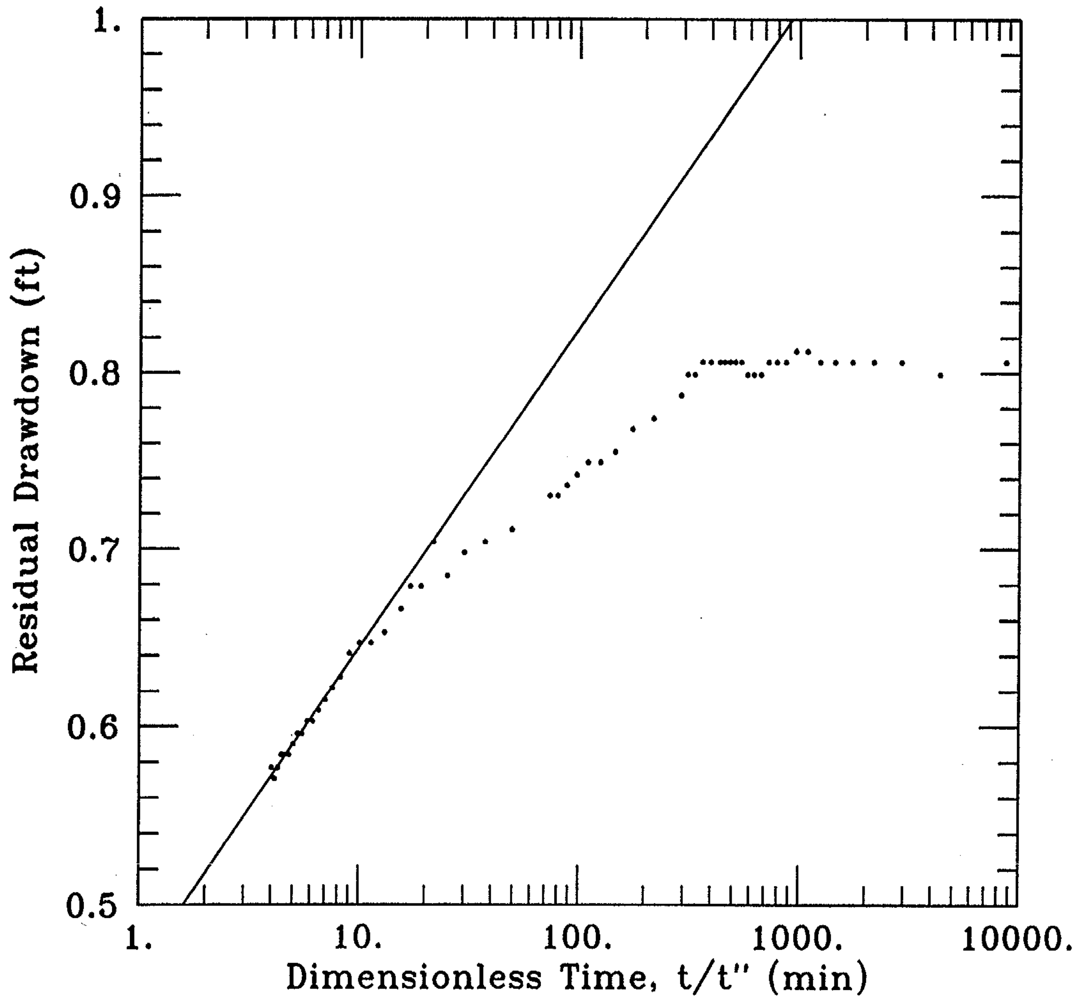
TEST DATA:
Q = 30. gal/min
r = 174.6 ft
r_c = 0.25 ft
r_w = 0.5 ft
b = 300. ft

PARAMETER ESTIMATES:
T = 1049.4 ft^2/day
S' = 1.653

AQTESOLV

Client: LANTDIV	Company: Baker Environmental, Inc.
Location: Site 82 -- MCB Camp Lejeune	Project: 62470-222

DP-1 RECOVERY DATA



DATA SET:
DP-1LR.DAT
07/11/96

AQUIFER MODEL:
Confined

SOLUTION METHOD:
Theis Recovery

PROJECT DATA:
test date: February 19-24, 1996
test well: DRW-1
obs. well: DP-1

TEST DATA:
Q = 30. gal/min
r = 251.7 ft
r_c = 0.25 ft
r_w = 0.5 ft
b = 300. ft

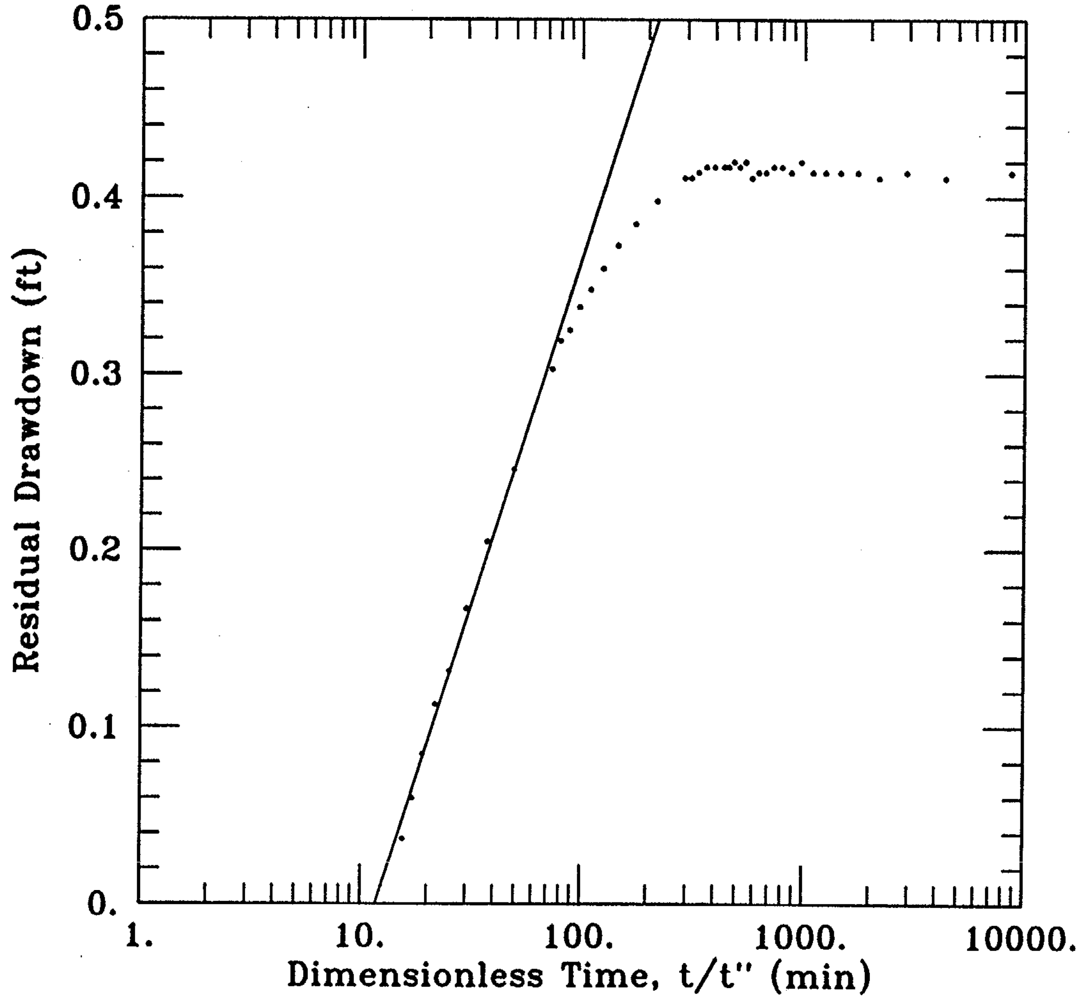
PARAMETER ESTIMATES:
T = 5063.5 ft^2/day
S' = 0.002778

AQTESOLV

Client: **LANTDIV** | Company: **Baker Environmental, Inc.**

Location: **Site 82 -- MCB Camp Lejeune** | Project: **62470-222**

6GW-15D RECOVERY DATA



DATA SET:
15D-LR.DAT
02/26/96

AQUIFER MODEL:
Confined

SOLUTION METHOD:
Theis Recovery

PROJECT DATA:
test date: February 19-24, 1996
test well: DRW-1
obs. well: 6GW-15D

TEST DATA:
Q = 30. gal/min
r = 379.2 ft
r_c= 0.25 ft
r_w= 0.5 ft
b = 300. ft

PARAMETER ESTIMATES:
T = 2340.6 ft^2/day
S' = 11.67

AQTESOLV

APPENDIX B
BRAGS MODFLOW AND MODPATH INPUT & OUTPUT FILES (CD-ROM)

APPENDIX C
SITE 82 MODFLOW AND MODPATH INPUT & OUTPUT FILES (CD-ROM)